EMBANKMENT QUALITY
PHASE I REPORT

Sponsored by the Project Development Division
of the Iowa Department of Transportation
and the Iowa Highway Research Board
Iowa DOT Project TR-401
CTRE Management Project 97-8

MAY 1998

CTRE
Center for Transportation Research and Education

IOWA STATE UNIVERSITY

Iowa Department of Transportation

EMBANKMENT QUALITY
PHASE I REPORT

Principal Investigator
Kenneth Bergeson
Professor of Civil Engineering
Iowa State University

Co-Principal Investigator
Chuck Jahren
Assistant Professor of Construction Engineering
Iowa State University

Graduate Research Assistants
Matt Wermager
Dave White

Sponsored by the Project Development Division
of the Iowa Department of Transportation
and the Iowa Highway Research Board
Iowa DOT Project TR-401

Preparation of this report was financed in part
through funds provided by the Iowa Department of Transportation
through its research management agreement with the
Center for Transportation Research and Education.
CTRE Management Project 97-8

Center for Transportation Research and Education
Iowa State University
Iowa State University Research Park
2625 North Loop Drive, Suite 2100
Ames, IA 50010-8615
Telephone: 515-294-8103
Fax: 515-294-0467
http://www.ctre.iastate.edu

May 1998

CTRE's mission is to develop and implement innovative methods, materials, and technologies for improving transportation efficiency, safety, and reliability, while enhancing the educational experience of students in transportation-related fields.

TABLE of CONTENTS

LIST OF FIGURES

LIST OF TABLES

ABSTRACT

Phase I was initiated as a result of internal Iowa DOT studies that raised concerns about the quality of embankments being constructed. Some large embankments have recently developed slope stability problems. In addition, pavement roughness has been noted shortly after roads were opened to traffic. This raised the question as to whether the current Iowa DOT embankment construction specifications are adequate. The primary objective of Phase I was to evaluate the quality of embankments being constructed under the current Iowa DOT specifications.

The project was initiated in May 1997 with a tour of several embankment projects being constructed around the state. At each of these projects the resident construction engineer, field inspector, and contractor were interviewed with respect to their opinion of the current specifications. From construction observations and discussion during these visits it became obvious that there were problems with the current embankment construction specifications. Six embankment projects were selected for in-depth analysis and to represent the full range of soil types being used across the state. The results of Phase I field and laboratory construction testing and observations and post construction testing are briefly summarized as follows.

Field Personnel (Iowa DOT and Contractors) Observations – Personnel appear to be generally conscientious and trying to do a good job but they are misidentifying soils ("unsuitable" and "class 10" soils being used as "select"), they lack soil identification skills (knowledge and equipment), and they are relying heavily on soil design plan sheets for determining unsuitable, suitable and select soils. Soils design data appear accurate, and are necessary, but spacing between borings and soil mixing during construction operations makes it difficult to differentiate the soils in the field.

Current Specifications – The current method of identifying unsuitable, suitable, and select soils may not be adequate. The one point Proctor does not appear adequate for identifying, or for field verification of compaction for all soils. The "sheepsfoot walkout" specification is not, for all soils, a reliable indicator of 1) degree of compaction, 2) compaction moisture content, and 3) adequate stability.

Construction Observations and Testing (Cohesive Soils) - The sheepsfoot walkout specification is producing embankments where soils are being placed wet of standard Proctor optimum moisture, compacted to near 100 percent saturation, and overcompacted resulting in an embankment that 1) has low soils shear strength (inadequate stability), 2) has a possibility of positive pore water pressure development (as embankment height increases) which results in a further reduction in shear strength, and 3) sets the stage for potential slope failure.

Construction Observations and Testing (Cohesionless Soils) - Compaction was being attempted with sheepsfoot rollers (vibratory necessary), and being monitored using standard Proctor testing which is an inappropriate method and can grossly overestimate degree of compaction.

Overall evaluation of the results of Phase I indicate that we are not consistently obtaining a quality embankment constructed under the current Iowa DOT specifications. Based on the foregoing, recommendations were made for Phase II to evaluate alternative specifications and develop rapid field methods for compaction control and soil identification.

INTRODUCTION

Embankments provide foundations for much of our transportation infrastructure. Quality construction is required to maintain smooth riding pavements and to provide slope stability. Proper selection of soil, adequate moisture control, and uniform compaction are required for a quality embankment. Internal Iowa DOT studies have raised concerns about the quality of embankments. Large embankments have occasionally exhibited slope stability problems. Resulting slides have caused encroachment on private property and damage to drainage structures. Pavement roughness has also been noted shortly after roads have been opened to traffic on projects that have been graded and paved in the same year. The costs for remediating such failures are high.

Soils available for embankment construction in Iowa generally range from A-4 soils, which are very fine sands and silts that are subject to frost heave, to A-6 and A-7 soils, which predominate across the state. The A-6 and A-7 groups include shrink/swell clayey soils. In general these soils rate from poor to fair in suitability as subgrade soils. Because of their abundance, economics dictate that these soils must be used on the projects even though they exhibit shrink/swell properties. Because these are marginal soils it is critical that the embankments be placed with proper compaction and moisture content.

Soils for embankment projects are identified during the exploration phase of the construction process. Borings are taken periodically along the proposed route and at potential borrow pits. The soils are tested to determine their engineering properties. Atterberg limits are determined and in-situ moisture and density are compared to standard proctor values that are calculated by the one point proctor method (Iowa Test Method Number 103-C). It is impossible, however, to completely and accurately characterize soils profiles because of the variability between boring locations. Therefore it is necessary for Iowa DOT field staff and contractors to be able to recognize that soil changes have occurred and make the proper field adjustments. The current Iowa DOT embankment specifications require sheepsfoot roller walkout as the acceptance method for compaction. Moisture control is not required except for subgrade treatment areas. Current practice requires good judgment and considerable experience on the part of the field personnel and contractors. Few field tests are available to the construction field staff. Many of these staff personnel and contractors gained their experience during the time of intense interstate highway construction of the 1960's and 1970's. Because these people are at or near the age of retirement, that experience and judgment is being lost to the industry; the consequences may be a reduction in quality under the current construction specifications.

Depending on roller configuration, soil moisture content, and soil type, soils may be under or overcompacted when the sheepsfoot roller walkout specification is used. If soil lifts are too thick, the "Oreo cookie effect" may result, where only the upper part of the lift is being compacted. If the soils are too wet, overcompaction from hauling equipment can occur with resultant shearing of the soil and building in shear planes within the embankment which can lead to slope failure.

The primary objective of Phase I was to evaluate the quality of embankments being constructed under the current specifications.

LITERATURE REVIEW

Construction with soil is one of the most complicated procedures in engineering. In no other field of engineering are there so many variables as to the material used for construction. It is also widely recognized that certain soils are much more suitable for some construction activities than others. Because of these two factors, it is essential that the field personnel know the properties of the specific soil being used. Observations of construction practice over the past 20 years led to the following conclusions (*1*).

- No new inspection procedures have been introduced except the introduction of nuclear density methods of determining in-place moisture and density.

- The percentage of soil tested is extremely small compared to the amount of soil placed. Thus, the compacted soil is accepted by relying heavily on the judgment of the inspector.

- The amount of testing conducted for compaction is usually insufficient. Thus, the testing that is completed is only for document certification or to guide the inspector's judgment.

A general understanding of soil and its different properties is essential for building a quality embankment. The engineering properties of a soil can vary greatly from gravels to clays. In order to build a quality embankment, the specific properties of the soil being used must be understood in order for proper field judgment to be made.

There is the constant debate among practitioners in geotechnical engineering, about whether to compact soils wet of optimum moisture content or dry of optimum moisture content. There is no decisive answer to this question. The only answer that is correct is that the ideal moisture content depends on material type and the desired characteristics (which often are competing) of the embankment. Strength, stability, density, low permeability, low shrink swell behavior, and low collapsibility are all desired outcomes of a quality embankment.

Strength is obviously a desirable characteristic and is a function of many factors but can be directly related to moisture content. The U.S. Army Corps of Engineers used California Bearing Ratio (CBR) as an efficient measurement of strength in cohesive materials. They reported, "The unsoaked CBR values are high on the dry side of optimum, but there is a dramatic loss in strength as molding moisture content is increased" (*2, 3*). Hilf (*4*) had the same results from tests using penetration resistance as a measure of strength. When a soil is in a dry stage, it exhibits high strength due to an appreciable interparticle attractive force created by a high curvature of the menisci between soil particles. Further wetting, however, greatly reduces this frictional strength by lubrication of the soil particles. Alternately, in cohesionless materials, the strength is not significantly affected by an increase in moisture content due to its high hydraulic conductivity.

Stability is a second desirable characteristic. Stability can not be defined as one characteristic, however. There is stability related to strength, which reacts to moisture content as described above. There is also volumetric stability. When dealing with highly plastic clays, this is an extremely important factor since they exhibit shrink/swell behavior with change in moisture content. Swelling of clays causes more damage in the United States than do the combined effects of all other natural disasters. It is general practice when dealing with fat clays to place the fill wet of optimum. This is basically forcing the clay to swell before compacting it in the embankment (*4, 5*). Moisture content becomes important in cohesionless materials with respect to volumetric stability when the bulking phenomenon is considered. At the bulking moisture content, a cohesionless soil will undergo volumetric expansion or "bulk." Additionally, the material will exhibit apparent cohesion which can resist any compactive effort. However, the addition of water will reduce the apparent cohesion and compaction can be achieved. Therefore, in terms of volumetric stability, a truly cohesionless material should be compacted dry or saturated (*6*).

Density is perhaps the characteristic most widely associated with embankment construction. The Proctor test is the most widely used laboratory test to determine maximum dry density and optimum moisture content of cohesive soils as a function of compaction energy. However, the standard Proctor test is not a valid test for cohesionless soils (*4, 7*). Cohesionless soils require the relative density test introduced by Terzaghi (*8*) to determine a maximum and minimum dry density.

Figure 1 represents desirable engineering characteristics of embankment soils and their relationship to Proctor moisture content and density.

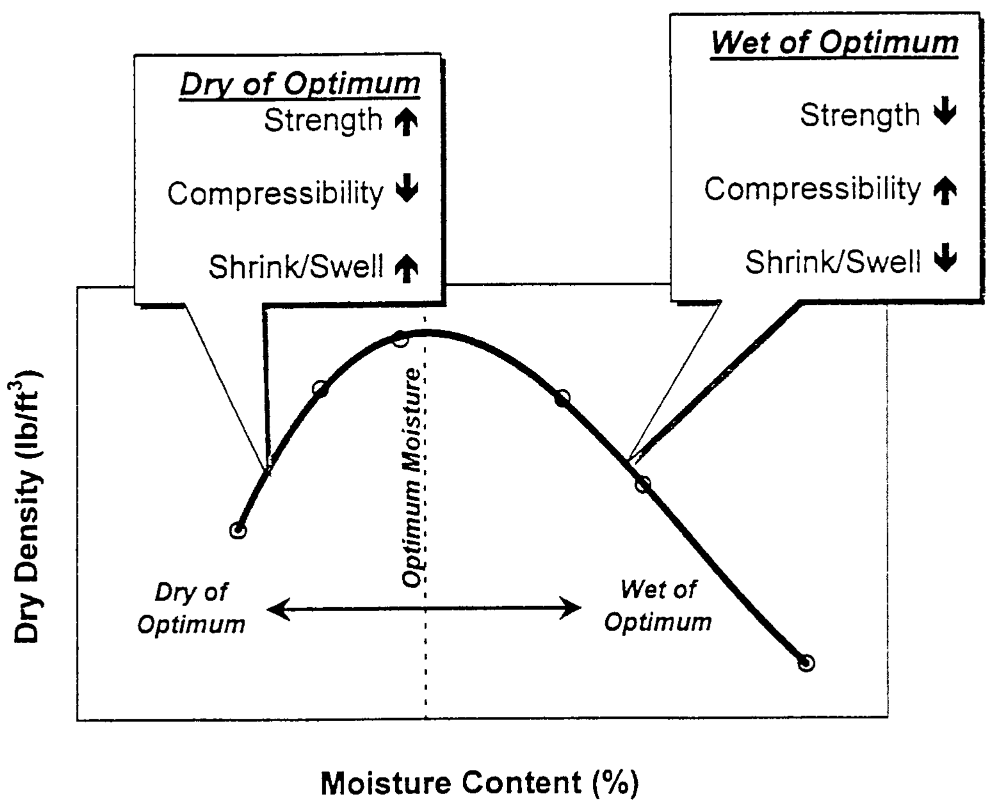


FIGURE 1 Engineering properties of soil on the wet and dry side of standard Proctor "Optimum" moisture content

Once the desirable material properties have been identified, the next process in building a quality embankment is the correct placement of the soil. The importance of soil preparation before rolling is not adequately appreciated. Blending of the soil to get a homogenous composition and moisture content is essential for quality embankment construction (*1*). Proper roller identification and use is also essential. Not all rollers are adequate for all soil types. Sheepsfoot rollers are ideal for cohesive soils while a vibratory roller must be used on cohesionless materials. Intergrade soils require intergrade rollers such as a vibratory sheepsfoot (*5*).

REVIEW OF THE EMBANKMENT SPECIFICATIONS OF THE 50 DOT'S

Research personnel investigated the embankment construction specifications of the U.S. DOT's. In particular, specifications on moisture content, density, lift thickness, and discing were investigated. These specifications were then compared to the Iowa DOT specifications.

Moisture Control Requirements

The current Iowa DOT specification does not require moisture content control on the embankment except for subgrade treatments. The specification calls for moisture control to provide for adequate density. This specification is similar for 31 other states. The specifications for the remaining 19 states required specific moisture control on the embankment. Table 1 shows the different moisture content requirements of all of the DOT's.

The specifications include a wide range of required moisture contents. Obviously, there is not a consistent philosophy as to what moisture content provides the best compaction and stability. Some states do not accept moisture contents above optimum; some do not accept moisture contents below optimum. As can be seen, 5 of the 19 DOT's that require moisture content control require the moisture to fall within ±2% of optimum.

Some DOT's moisture control requirements were dependent on the materials being used. For example, Kansas has five different moisture content ranges depending on the soil classification. New Mexico bases the moisture content range on the plasticity of the soil. Figure 2 shows the geographic location of those states requiring moisture control. It is important to note that many of the states requiring moisture control are in mid to upper United States regions where environmental conditions are problematic for highway performance.

TABLE 1 Moisture control specifications for embankment of soil fill as reported by state DOT's

Moisture Control Specification	Number of States
Adequate moisture to achieve specified compaction	**31**
±5	1
-4 to 0	1
-4 to +2	3
-4 to +5	1
±3	1
-2 to 0	1
-2 to +1	1
±2	5
0 to +3	1
0 to +5	1
≤ +2	1
≤ +3	1
≤ 115% of Optimum	1

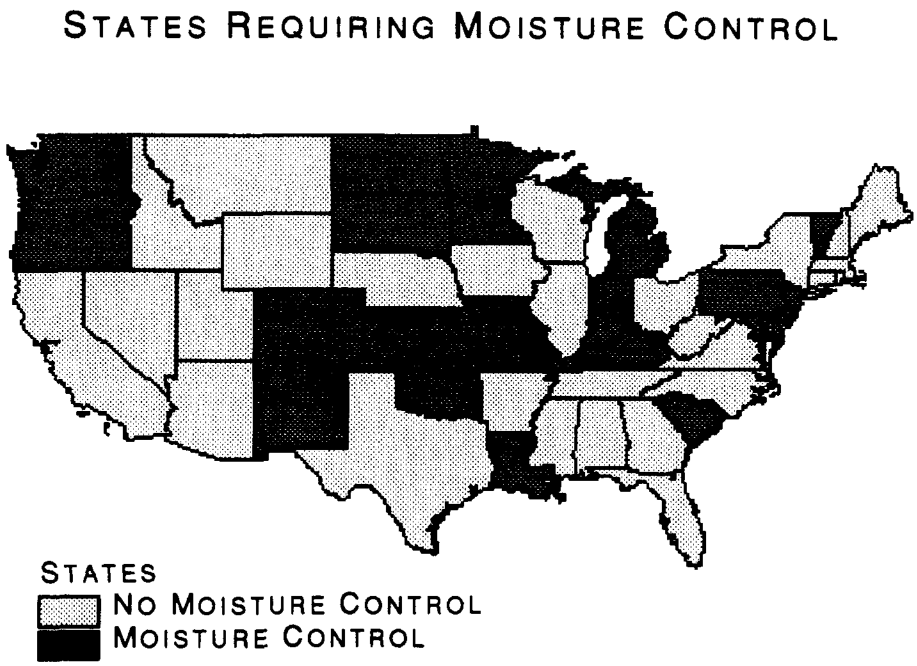


FIGURE 2 DOT's requiring moisture control

Density Control

The current Iowa DOT specification for density control requires sheepsfoot walkout for achieving adequate compaction of an embankment. Minimal in-situ quality control testing is performed to find the in-place density except for the subgrade treatments. As can be seen from Table 2, a minimum of 95 percent of standard Proctor maximum density is the specification used by the majority of the states. Ten states specify modified Proctor requirements.

Table 2 Density control specifications for soil fill embankment as reported by state DOT's

Density Control		
Relative Compaction Limit (%)	**No. of States Standard Effort**	**No. of States Modified Effort**
85	0	1
90	2	1
92	2	1
95	30	5
96	1	0
97	1	0
98	0	1
100	5	1

The Iowa specification differs from the other states in that the only in-situ testing for compaction is the roller walkout. The majority of the other states use in house testing methods, AASHTO T 191 (Density of Soil In-Place by the Sand-Cone Method), and T 238 (Density of Soil and Soil-Aggregate In-Place by Nuclear Methods) for compaction control. The only other state that stipulates roller walkout as part of the specification is Kansas; however, they use the walkout as one of their field tests for moisture content and not for density.

Similar to moisture control, many states utilize different density specifications for different types of materials. For example, Colorado requires 100 percent of maximum density (AASHTO T-99) for A-1, A-3, A-2-4, and A-2-5 soils -- all other soils require 95 percent compaction.

Lift Thickness

The current Iowa DOT specification for lift thickness is 200mm (8 inches). This is comparable to the majority of the other DOT's. Table 3 shows the different lift thickness for the DOT's throughout the United States.

TABLE 3 Lift thickness specification for soil fill embankment as reported by state DOT's

Lift Thickness (mm)	Number of States
200 (8″)	42
230 (9″)	1
250 (10″)	1
300 (12″)	5
Dependent on compaction equipment	1

Foundation Preparation

Currently, the Iowa DOT does not require discing of embankment foundations. This is the case with many of the states. However, 19 states do require that the foundation of the embankment be disced or scarified before any embankment is placed, regardless of the embankment height. Many states additionally stipulate that any topsoil encountered be removed and stockpiled for dressing the embankment.

Other Specifications Reviewed

In the review of the state DOT's specifications, one reoccurring specification was of particular interest. The specification required the use of test lots and control strips. Basically, anytime a new soil was being placed in the embankment; a control strip or test lot would be built out of the new material. Strict moisture and density control would be used to build this and the average density of the control strip or test would be the target density for the rest of the embankment built out of this material. A geographic representation of the eight states currently using this method is shown in Figure 3.

Other specifications encountered in the review of the DOT's included use of:

- Settlement gauges and rods
- Proofrolling prior to final trimming
- Topsoil dressing on all soils
- Discing of each embankment layer
- Maximum length of embankment construction (Arkansas – 200 feet, Florida – 300 feet for example)

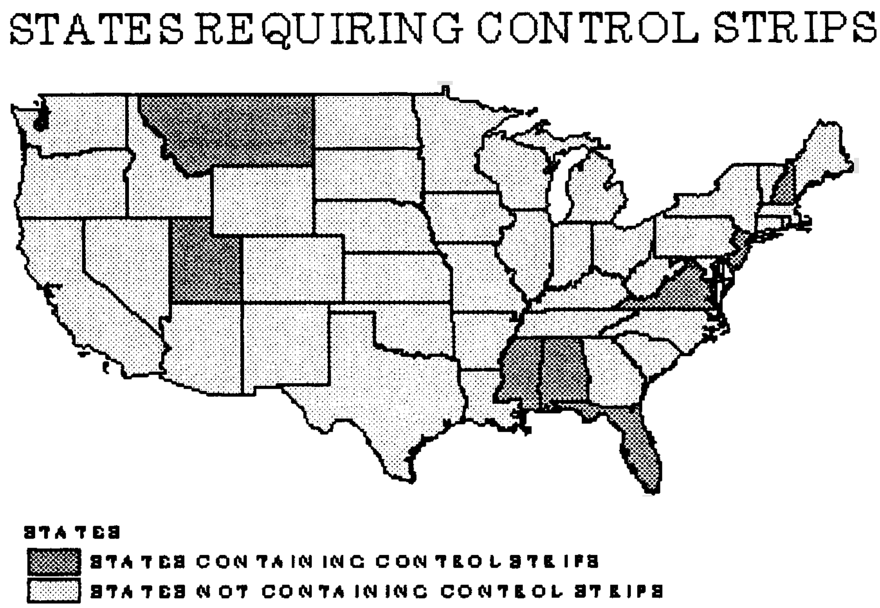


FIGURE 3 DOT's requiring control strips

Unsuitable Soils Practice

The disposal of soils determined to be unsuitable was the last aspect of the embankment specification that was investigated. For disposal practice, only the Midwest states were investigated. The current Iowa specification classifies unsuitable soils as having a group index over 30, having a unit weight less than 95 pcf, and having more than 3 percent carbon. Soils classified as unsuitable by the above criteria must be placed a stipulated distance below subgrade elevation. Iowa has the most extensive specification in terms of unsuitable soil disposal. Most states, such as Oklahoma, North Dakota, Minnesota, Kansas, Indiana, and Illinois, do not have criteria for classifying soil as unsuitable except by the discretion of the engineer. Some states, however, do have classification criteria. For example, Ohio classifies a soil as unsuitable if the unit weight is less than 90 pcf or if the liquid limit is above 65 while Michigan classifies unsuitable soil as having a unit weight less than 95 pcf.

In terms of placement of the unsuitable soil, most states do not allow unsuitable soil to be used in the embankment. These states are Oklahoma, North Dakota, Nebraska, Kansas, Indiana, and Illinois. Minnesota will allow some unsuitables to be placed in the embankment in accordance with special provisions of the specific contract as long as the soil is 3 feet below subgrade. Missouri will allow unsuitables on the side slopes two feet below finished shoulder elevation. Wisconsin allows unsuitable material to flatten slopes and fill low places in the right of way. Finally, Ohio and Michigan only allow silts to be placed 3 feet below subgrade.

PHASE I PROCEDURE

The project steering committee consisted of Iowa DOT managing technical representative John Vu from the Construction office, Bob Stanley from Soils Design, Dave Woofter, Ottumwa Construction office, Bob Jimerson, Creston Construction office, Jerry Danforth, Cedar Rapids Construction office, and John Moyna, C.J. Moyna and Sons, Inc.

The project was initiated in May 1997 with a tour, led by John Vu, of several embankment projects being constructed around the state. At each of these projects the resident construction engineer, field inspector, and contractor were interviewed with respect to their opinion of the current specifications. From construction observations and discussion during these visits it became obvious that there were problems with the current embankment construction specifications.

The initial intent of Task 4 of the Phase I proposal was primarily observational in nature as follows:

> ***Task 4 – Observation of current practice –*** Researchers will make field trips to observe embankment compaction procedures at six projects. This will provide an understanding of the current practice. During the field trips, researchers will interview Iowa DOT field staff and contractor personnel to find their opinions about the advantages and disadvantages of the current system. Researchers will also request suggestions for improvement.

Following field visits and discussions with John Vu and the steering committee, it was decided that in order to adequately evaluate the quality of the embankments being constructed hard data were needed to verify visual observations. A significant portion of the research effort in Phase I was redirected toward Task 4 with the inclusion of a significant amount of field embankment construction (particularly moisture content, density, and soils identification testing), and laboratory testing and post construction testing of embankment properties.

A considerable amount of the research effort was also directed toward Task 6 with the inclusion of field and laboratory testing of various methods for classifying and testing soils.

> ***Task 6 – Assessment methods to rapidly field classify soils, and test soils for compaction moisture and density –*** An assessment of currently available equipment that is low cost and portable will be conducted. Recommendations on equipment and test procedure guidelines will be developed.

Six embankment projects were selected for in-depth analysis to represent the full range of soil types being used across the state. The projects investigated, along with predominant soil types, are shown in Table 4.

TABLE 4 Embankment projects investigated in Phase I

City	Project	Predominant Embankment Soil Type
1. Mason City	Relocation of US 18 – I 35 East to Raven Avenue	Fat Clays to Lean Clays
2. Waverly	Relocation of US 218	Lean Clays
3. Sioux City	Relocation of US 75	Clayey Silts
4. Monroe	Highway 163 Bypass	Fat Clays
5. Mason City	Relocation of US 18 - Raven Avenue to Winnebago River	Sands
6. Prairie City	Highway 163 Bypass	Fat Clays

FIELD INVESTIGATION RESULTS AND DISCUSSION

In order to evaluate the current field practice, ISU research personnel conducted field monitoring and testing. Activities included observations of fill placement and in-situ moisture and density testing. Upon completion of the embankments, subsurface explorations were performed at selected locations to obtain information on actual finished conditions and to develop an engineering evaluation for each project. The investigation procedures and results of the testing and evaluation are described herein.

Field Testing Procedures

In-situ lift-by-lift field density and moisture tests were performed on a variety of fill materials being placed in the embankments. Three methods were utilized to obtain the field density information: the nuclear density gauge, Shelby tubes, and US Army Corps of Engineers Surface Soil Sampler. Most tests were performed using the US Army Corps of Engineers Surface Soil Sampler, which was developed to take density or compaction tests at or near the ground surface. The density sampler consists of a 10-lb drop hammer and thin walled steel tubes (Figure 4) machined to a calibrated volume in accordance with ASTM Tests Designation D-2937. The steel tubes were driven into the compacted soil then removed, trimmed and weighed to obtain wet density. A moisture content sample was then obtained from the center of each tube.

Nuclear field density tests were performed with a Humbolt model 5001 nuclear density gauge using the direct transmission method. Tests were conducted in accordance with ASTM Tests Designation D-2922 and D-3017 for compaction testing of soils.

FIGURE 4 US Army Corps of Engineers surface soil sampler

Moisture-density relationship tests (ASTM D-698) were performed on several samples of material. These relationships were used to determine percent compaction and reference moisture contents. Results of the field density tests are indicated in Appendix A for the corresponding project location.

Drilling and Sampling Procedures

Once the embankments were near completion or at design subgrade elevation the subsurface explorations were conducted. The primary objective was to determine the in-situ conditions of the embankment materials after construction and to analyze these conditions, compared to construction test results, as they relate to embankment quality. The borings were performed by the Iowa DOT with a truck-mounted, rotary drill rig using hollow stem augers to advance the boreholes. Thin-wall tube and split-barrel sampling procedures (ASTM Specifications D-1587 and D-1586) were used to obtain representative soil samples. The thin-walled tube sampling procedure utilized a seamless steel tube with a sharp cutting edge that was pushed hydraulically into the soil to obtain relatively undisturbed samples of cohesive or moderately cohesive soil. For cohesionless soil the sampling procedure utilized a standard 2-inch O.D. split-barrel sampler that was driven into the soil with a 140-pound hammer falling a distance of 30 inches. The number of blows required to advance the sampler the last 12 inches of the 18-inch penetration was recorded as the standard penetration resistance (SPT) value.

The samples were sealed and returned to the laboratory for further examination, classification, and testing. Unconfined compression, moisture content, and density tests were performed on representative portions of the undisturbed samples obtained by the thin-wall sampler. A calibrated hand penetrometer was used to determine the approximate unconfined compressive strength when samples were deformed or of insufficient size. Atterberg Limits tests were also conducted on samples representative of the soil. The results of the laboratory tests are presented on the respective Boring Logs in Appendix B.

Project Locations and Description

Relocation of US Highway 18 - Cerro Gordo County - From I-35 to Just East. of Eisenhower Avenue

Between August 5, and October 19, 1997, observations and field density tests were performed on fill material placed near Sta. 2177+00 on the east bridge berm of the I-35 bridge crossing. Field tests and observations were conducted from approximately 10 to 20 feet below design subgrade elevation. At this location the fill materials consisted of a heterogeneous mixture of silts and clays with some sands and gravels. The majority of this structural fill was classified as A-4 and A-7-5 by the AASHTO classification system and SC (clayey sand) and CH (fat clay) by the Unified classification system. During fill placement, some of the on-site fill materials from borrow cuts were observed to contain moisture contents above optimum. In order to alter the moisture content of the fill material, periodically the contractor aerated the fill by discing and allowing to air dry. Compaction was achieved utilizing a sheepsfoot roller.

Field density tests indicated that percent compaction ranged from approximately 89% to over 100% of the standard Proctor with moisture content ranging from +0.4% to +5.0% above optimum. With respect to the average moisture-density relationship and the zero air-void curve, the field density tests are plotted as shown in Figure 5. A general acceptance range of ±2.0 % of the standard Proctor optimum moisture content, and 95 percent of standard Proctor density was selected for comparison.

The relationship shown in Figure 5 indicates that the majority of fill material tested was wet of optimum and being compacted to near 100 percent saturation. When the soil is saturated no air voids are left in the material and, with additional load or additional compaction, shear failure may occur. This is a first step in setting the stage for slope failure to occur.

On November 11, 1997, after the embankment was near completion, boring B-1 was conducted through the embankment where the field observations and testing had taken place. The boring depth was approximately 24 feet deep from the top of embankment. The soil conditions encountered consisted primarily of silty clay with sand and gravel. Auger refusal was encountered near the depth of the natural foundation soils. In order to compare the density of the soil during placement and after the embankment was constructed density tests were performed on the samples obtained from the borings. The results of the density tests after embankment construction are shown in Figure 6. The moisture contents ranged from +0.2% to +18.5% above optimum moisture content and the percent compaction ranged from approximately 73 percent to over 100 percent. These test results are similar to the field test results during construction, indicating the material was near saturation and approaching the zero air void curve. Near the foundation of the embankment, test results indicated that the structural fill material was approximately 13 to 18 percent above optimum moisture content with very low densities. The depths are shown on boring B-1 in Appendix B. Boring B-1 consisted of 2 feet of silty clay underlain by 11 feet of gray fat clay with silt and the bottom 9 feet consisting of a mixture of clays, silts, and some sand. A majority the structural fill material encountered in the borings had a medium stiff to stiff consistency. Isolated soft layers were encountered throughout the fill and directly above the foundation. The soil classifications and Atterberg limits of the soils encountered in boring B-1 are shown on Figure 7. The Liquid Limit varied from 22 percent at the surface to 56 percent at 5.3 feet below top of grade and the respective Plasticity Indexes were 8 percent and 29 percent. These high moisture content, low-density, low strength soils near the foundation are subject to variable settlement due to consolidation and add to the potential of slope failure.

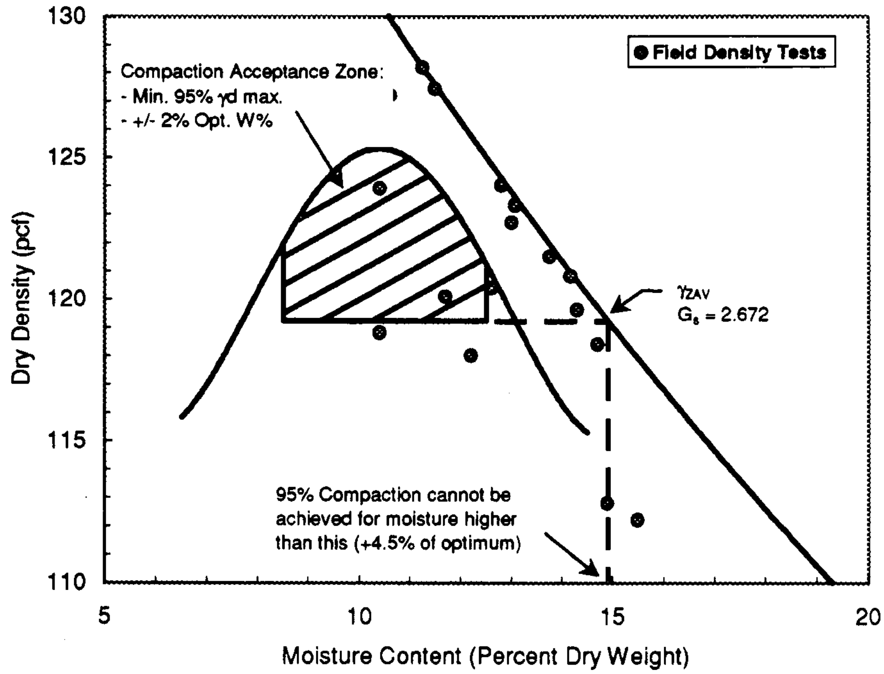


FIGURE 5 US Highway 18 Relocation - Field density results during construction with corresponding moisture-density relationship and zero air-void curve

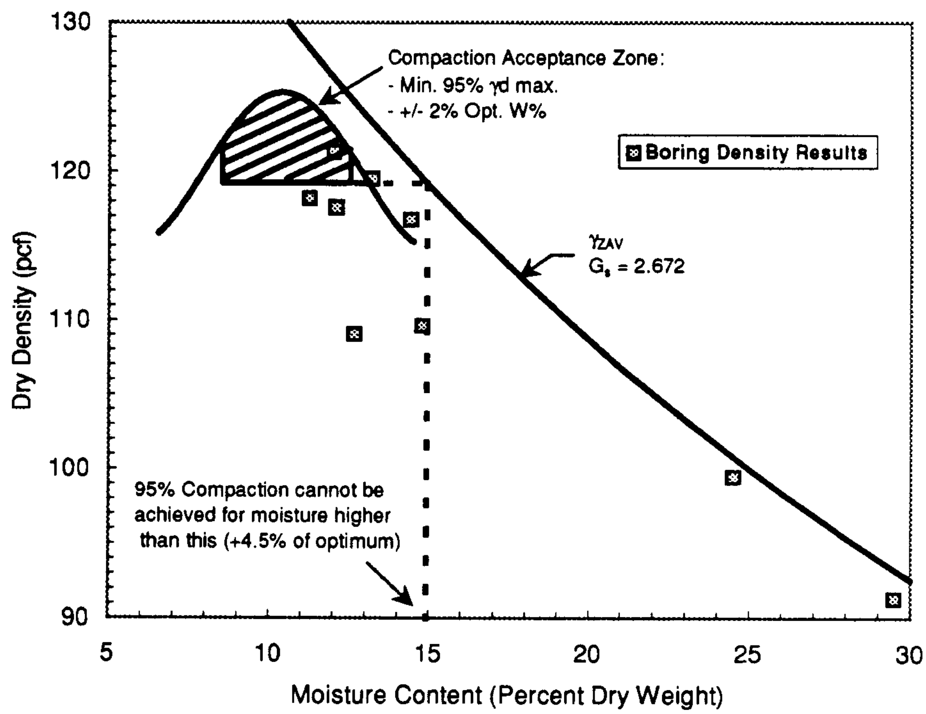
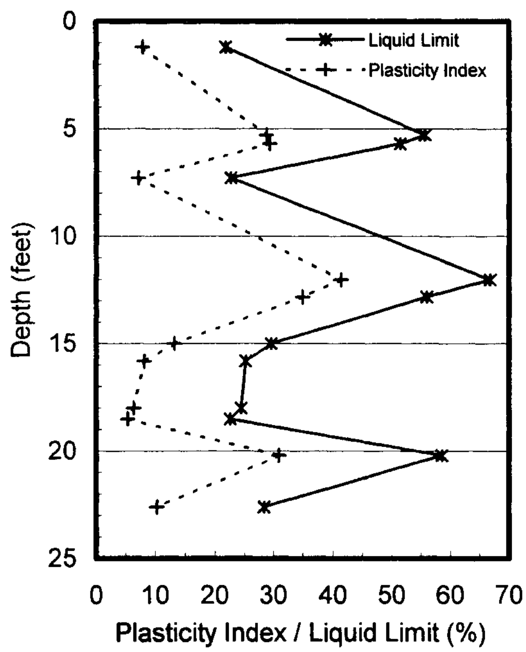


FIGURE 6 US Highway 18 Relocation - Boring density results after embankment construction with corresponding moisture-density relationship and zero air-void curve



Depth (Feet)	AASHTO Classification	Unified Classification
1.2-1.7	A-4	SC
5.3-5.7	A-7-5(33)	CH
5.7-6.1	A-7-5(33)	CH
12.0-12.5	A-7-5	CH
12.8-13.5	A-7-5	CH
15.0-15.8	A-4/A-6	SC
15.8-16.3	A-4	SC
18.0-18.5	A-4	SC
18.5-19.0	A-2-4	SM-SC
20.2-20.7	A-7-5(29)	CH
22.6-23.1	A-1	SM

FIGURE 7 Atterberg limits with soil classification profile for boring B-1

Relocation of US Highway 18 - Cerro Gordo County – From Raven Avenue East to Winnebago River in Floyd County.

Just east of Raven Avenue field observations and a subsurface exploration were completed on an embankment, which consisted mostly of sands and gravels. On April 29, 1997 (before the start of this project) the Iowa DOT and an independent testing consultant performed several density tests on the sand and gravel embankment fill material. A nuclear density gauge was used by the consultant and the Iowa DOT to determine the in-situ density. These densities were compared to the standard Proctor maximum dry density to determine percent compaction. However, the standard Proctor is the incorrect laboratory method for determination of maximum dry density of granular materials. The percent compaction should be based on the relative density (D_r) (ASTM Tests Designation D-2049) not the standard Proctor. The in-situ density tests are shown as percent relative density on Figure 8 and indicate that the relative density ranged from approximately 55 to 87 percent which is in a medium compact to compact state. Typically a granular material specification would require a very compact relative density. The relative density (D_r) should range from 90 to 100 percent. In addition to the nuclear field density tests, the maximum dry density value obtained from the standard Proctor test is plotted on Figure 8. This correlates to a relative density (D_r) of only 46 percent, well below the desired 90 percent, but based on standard Proctor tests compaction results were reported in excess of 100 percent.

After completion of the embankment, boring B-2 was conducted on November 19, 1998. Standard penetration resistance testing (SPT) was used to determine in-situ relative density of the finished embankment. The blow counts per foot were measured approximately every other foot through the entire embankment to the foundation. As shown in Figure 9, the relative density varied with depth. Near the surface the density was high and decreased with depth. The low relative densities obtained from the SPT at the middle to lower region of the embankment were comparable to the field density tests taken during construction. The high densities near the surface could be a result of densification from construction traffic.

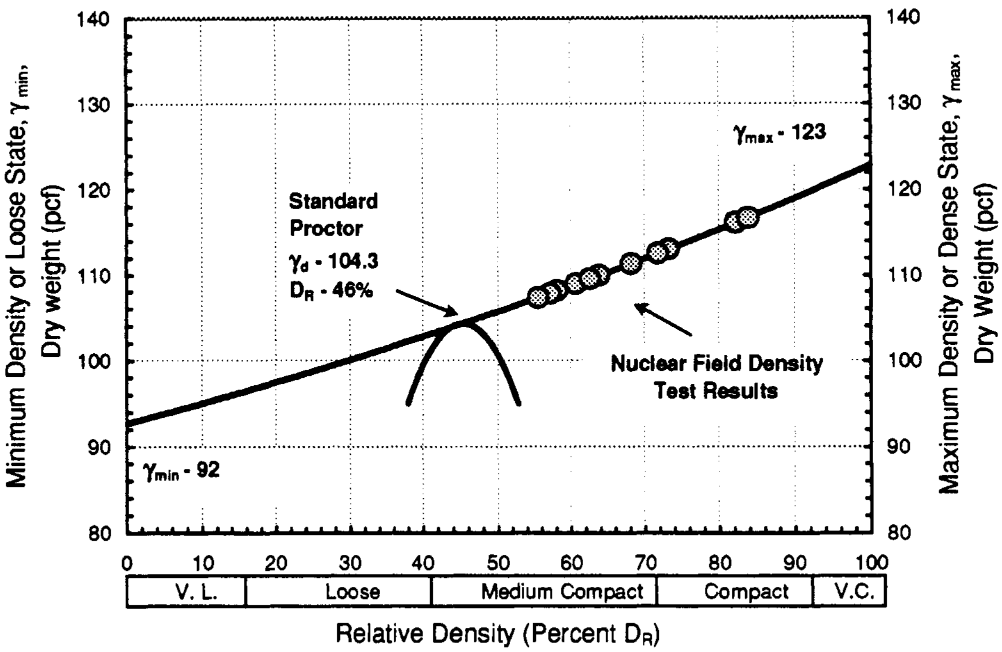


FIGURE 8 US Highway 18 Relocation - Relative density results for field density tests during embankment construction for granular structural fills

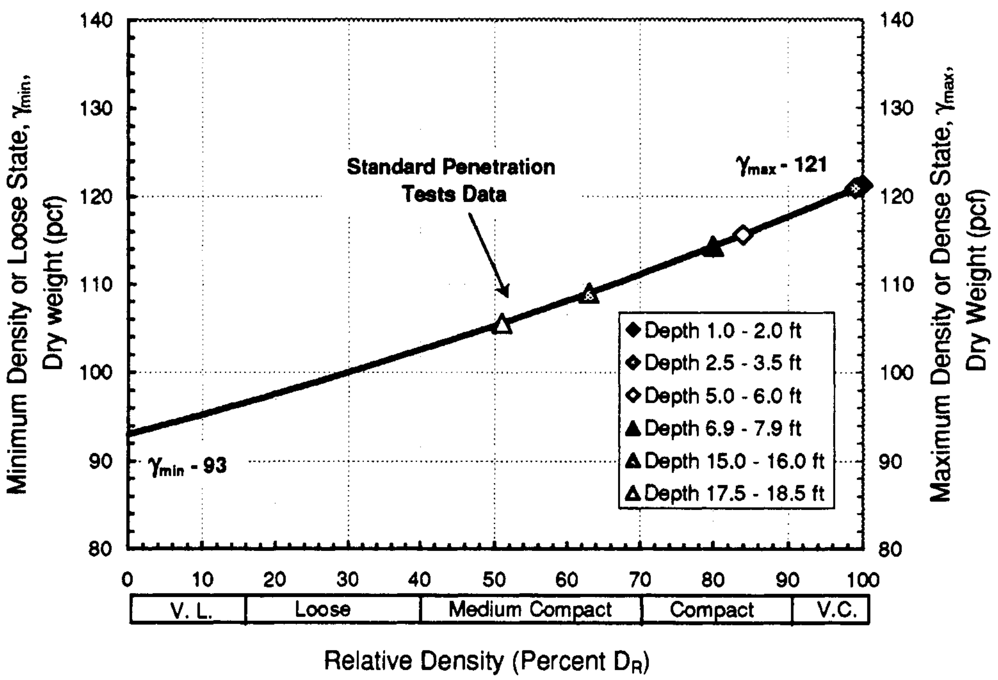


FIGURE 9 US Highway 18 Relocation - Relative density results from standard penetration resistance (SPT) data out of boring B-2 post embankment construction

US Highway 163 Bypass – Jasper County, Monroe

During construction of the north lane of US Highway 163 near Station 264+00, field density tests were performed on the fill material from approximately 5 to 17 feet below design subgrade elevation. Tests and observations were performed from June 29 to September 29, 1997. At this location the embankment was approximately 23 feet high and was constructed with fat clays from borrow and cut excavations. The structural fill material was generally classified by the ASSHTO classification system as A-7-5 and by the Unified classification system as CH (fat clay). The fill material for the entire depth of the embankment was fairly homogeneous with respect to material type.

Due to wet borrow materials and weather conditions much of the material was placed and compacted wet of optimum. The field density tests indicate that the moisture contents varied from +1.1% to +12.1% above optimum. Percent compaction ranged from 84 to 100 percent of standard Proctor. In respect to the average moisture-density relationship and the zero air-void curve, the field density tests are plotted as shown in Figure 10. Most of the soil is wet and under compacted. Much of the material is compacted to a density approaching the zero air void curve. Almost half of the field density tests indicate that soils have moisture contents 7 percent above optimum. Again this embankment can be expected to exhibit differential settlement and the stage has been set for potential slope failure.

On November 17, 1997 boring B-3 was conducted through the constructed embankment. The boring depth was approximately 20 feet from the top of subgrade. The soil was a fat clay for most of the boring. The post construction density results are shown in Figure 11. The moisture contents ranged from +1.4% to +8.2% above optimum while percent compaction varied from 81 to 99 percent of standard Proctor maximum dry density. Again as with the construction test results, a majority of samples tested were too wet to be compacted above the 95 percent compaction level.

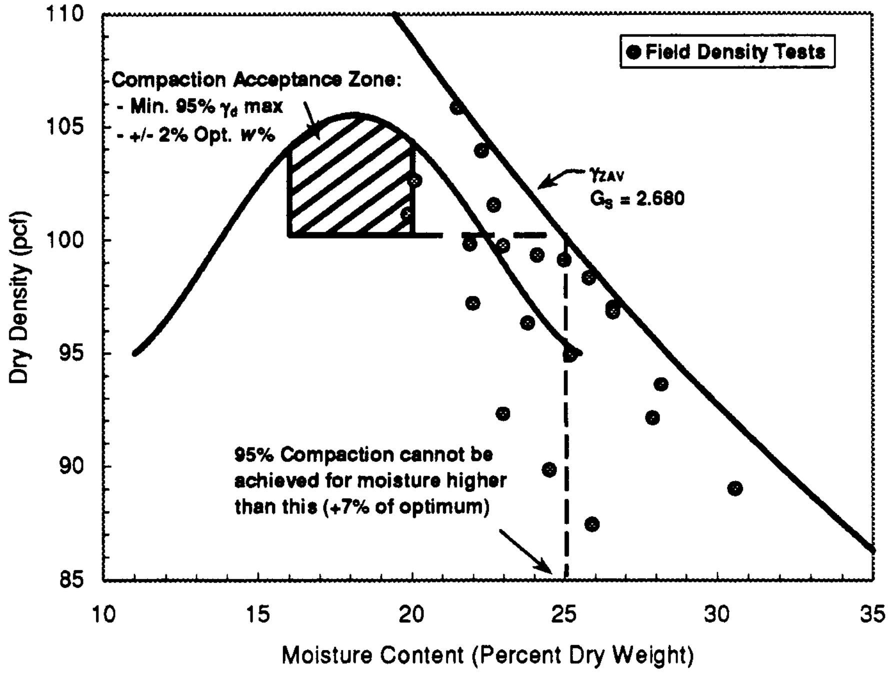


FIGURE 10 US Highway 163 Bypass, Monroe City - Field density results during construction with corresponding moisture-density relationship and zero air-void curve

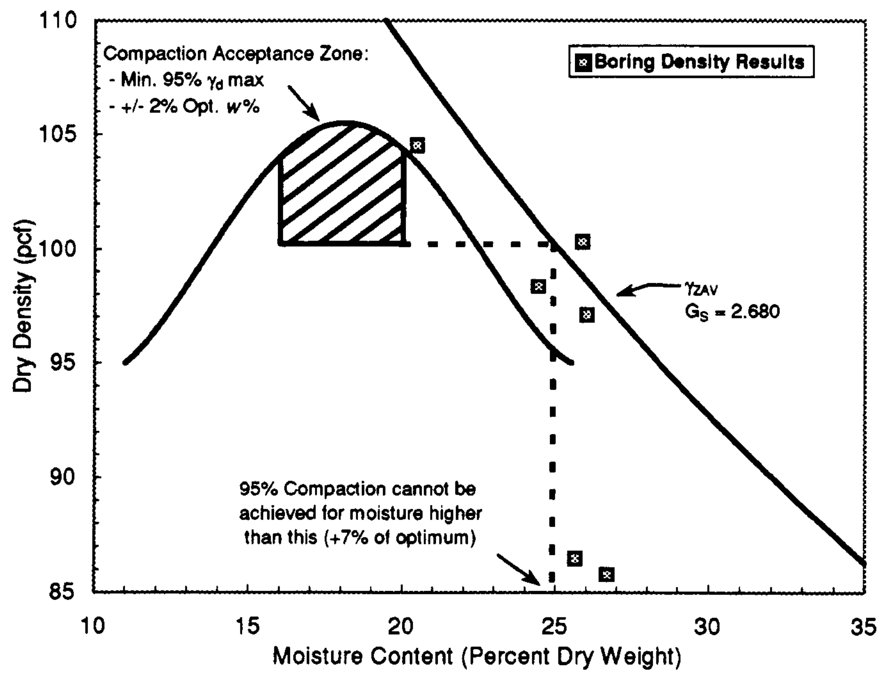
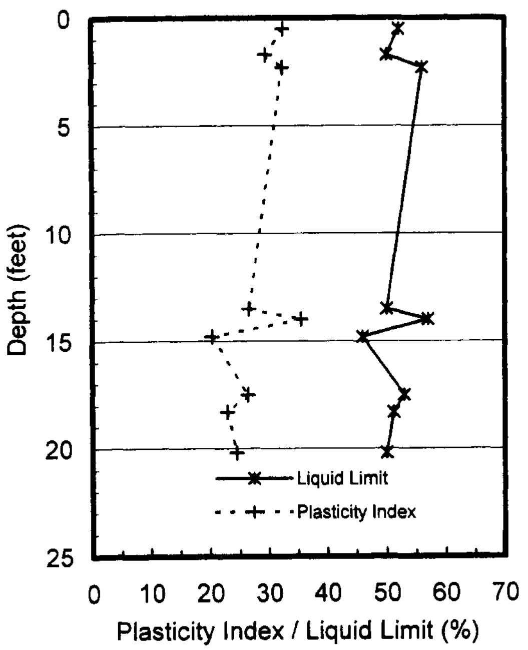


FIGURE 11 US Highway 163 Bypass, Monroe City - Boring density results after embankment construction with corresponding moisture-density relationship and zero air-void curve

Boring B-3 consisted of 17 feet fat clay underlain by black/brown clay with organic debris, which was believed to be the foundation material. Based on the unconfined compressive strength results most of the soil encountered in the boring had stiff to very stiff consistencies. The soil classifications and Atterberg limits of the soils encountered in boring B-3 are shown on Figure 12. The majority of the structural fill was characterized as unsuitable soil, which according to the current disposal specification is to be placed at least 5 feet below design subgrade elevation. As can be seen from the soil classification for boring B-3, unsuitable soils are present from the surface to approximately 14 feet below grade. Iowa DOT specifications require unsuitables to be disposed of 3-5 feet below subgrade surface elevation. Field personnel had not recognized this as an unsuitable soil and were using it as select material. The Liquid Limit near the surface was 52 percent with a Plastic Index of 32 percent. These soil types have a high affinity for water and will shrink and swell with a change in moisture content. Consequently a rough pavement may develop.



Depth (Feet)	AASHTO Classification	Unified Classification
0.5-1.0	A-7-6(34)	CH
1.7-2.3	A-7-5(32)	CH
2.3-2.8	A-7-5(37)	CH
13.5-14.0	A-7-5(30)	CH
14.0-14.7	A-7-5(38)	CH
14.8-15.5	A-7-5(23)	CL
17.5-18.0	A-7-5(30)	CH
18.3-19.0	A-7-5(27)	CH
20.2-20.7	A-7-5	CH

FIGURE 12 Atterberg limits with soil classification profile for boring B-3

Relocation of US Highway 218 - Bremer County - South of Waverly Northeast to 240th Street

On September 14, 1997, field density tests and observations were performed near station 480+00 on centerline of the proposed relocation of US 218. Observations and testing were performed from 18 to 20 feet below design subgrade elevation. At this location the embankment material was end dumped from trucks, manipulated with a bulldozer to form 9 to 12 inch lifts (greater than allowed by specifications), and then compacted with a large sheepsfoot roller. The fill material generally consisted of A-6 material as classified by the AASHTO classification system and as CL (lean clay) by the Unified classification system. Much of the soil contained a trace sand fraction with cobbles up to 10 inches in diameter. Based on visual observations the moisture content of the fill material appeared to be near optimum during placement. Field density tests indicate that the percent compaction ranged from 97 percent to over 100 percent of the standard Proctor with moisture contents ranging from -2.1% to +4.4% from optimum.

The field density tests are shown on Figure 13 with the moisture-density relationship and zero air-void curve. The results show that the fill material was placed near optimum moisture content and that adequate compaction was achieved. However, evidence of near overcompaction is present in at least one test. For the most part the fill material at this location was considered a good "Class 10" soil that was placed and compacted under the current sheepsfoot walkout specification.

After completion of the embankment, boring B-4 was drilled on November 20, 1997. The boring depth was approximately 20 feet from the top of embankment to auger refusal. At this location three separate borings were attempted due to auger refusal.

The soil conditions encountered consisted essentially of all lean clay with sand and cobbles. The post construction density results are shown in Figure 14. The moisture contents ranged from -2.5% to +6.6% from optimum while percent compaction varied from 92 percent to over 100 percent of standard Proctor maximum dry density. These results correlate well with the construction testing results, indicating in this case that the sheepsfoot walkout specification was adequate.

The fill material encountered in boring B-4 was fairly homogeneous throughout the embankment and had a very stiff to hard consistency. The soil classification and Atterberg limits of the soil encountered in boring B-4 are shown in Figure 15. Again this material was generally accepted as a good "Class 10" fill material and based on the field results should perform adequately under the pavement loads. Throughout the embankment the Liquid Limit varied from 24 to 36 percent with respective Plasticity Indices of 10 percent and 19 percent.

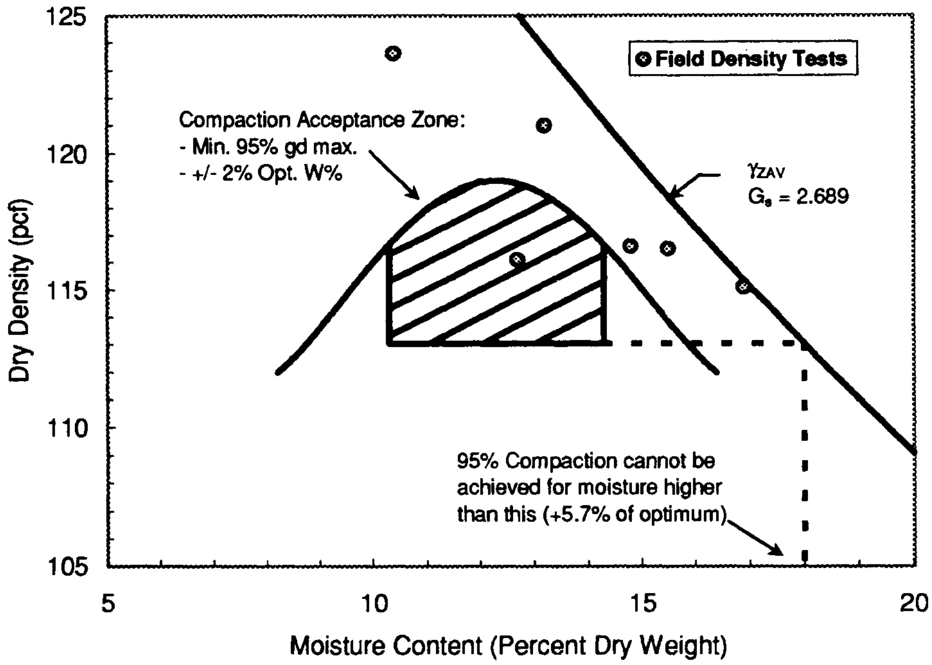


FIGURE 13 US Highway 218 - Field density results during construction with corresponding moisture-density relationship and zero air-void curve

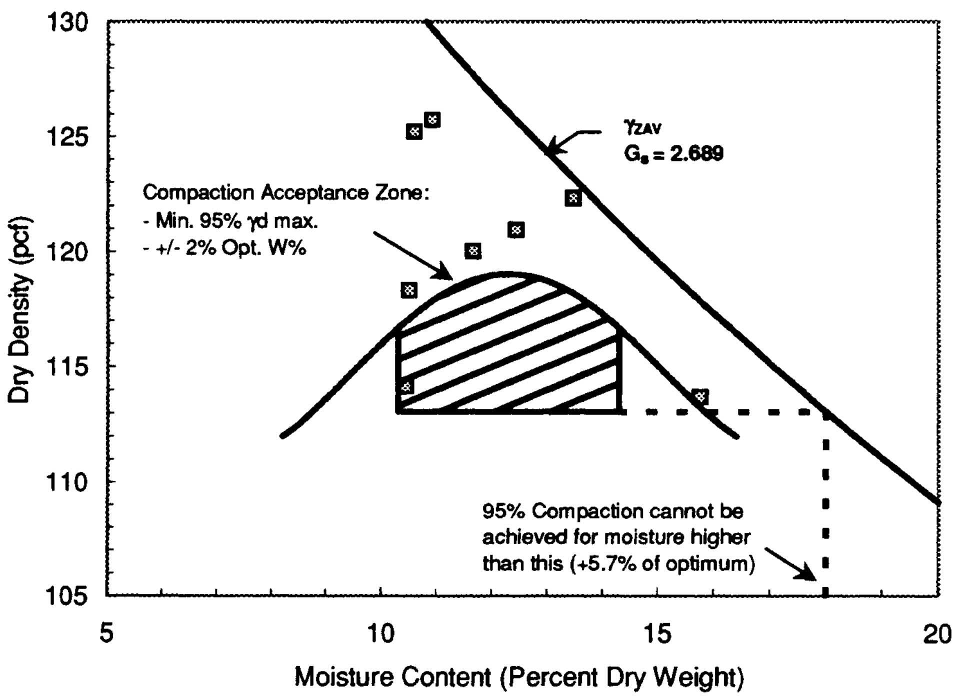
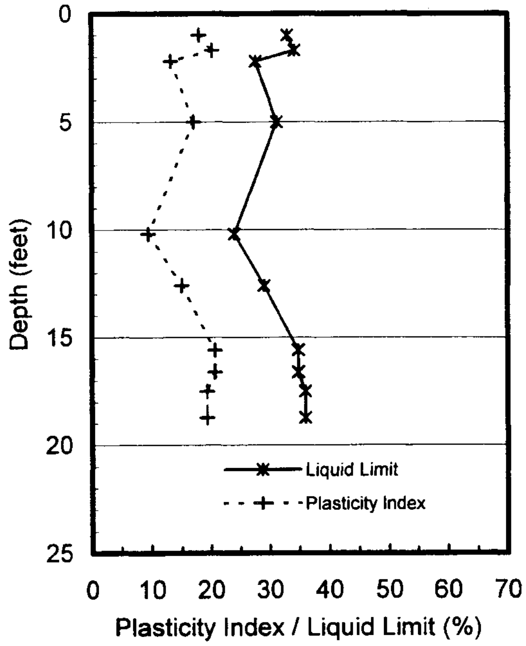


FIGURE 14 US Highway 218 - Boring density results after embankment construction with corresponding moisture-density relationship and zero air-void curve



Depth (Feet)	AASHTO Classification	Unified Classification
1.0-1.6	A-6	CL
1.7-2.2	A-6	CL
2.2-3.0	A-6	SC
5.0-5.5	A-6	SC
10.2-10.7	A-4	CL
12.6-13.1	A-6	CL/SC
15.6-16.1	A-6	CL
16.6-17.1	A-6	CL
17.5-18.0	A-6	CL
18.7-19.2	A-6	CL

FIGURE 15 Atterberg limits with soil classification profile for boring B-4

US Highway 163 Bypass - Jasper County - West of Prairie City to East of Prairie City

Between June 24 and August 12, 1997 field observations were made during construction of the north bridge berm crossing the proposed Highway 163 at station 1279+00. Observations of soil type, lift thickness, soil placement and compaction methods were recorded. The fill material at this location consisted of a brown/tan lean to fat clay with a trace of sand, which was excavated from an adjacent borrow pit. At the surface the embankment was to be capped with 2 feet of select material. The moisture content of the fill material in the lower portion of the embankment appeared to be well above optimum while the upper portion of the embankment appeared to have moisture contents close to optimum.

During fill placement it was observed that the loose lift thickness varied from 9 to 24 inches. The fill material was excavated and transported to the fill site with large scrapers. Some of the fill material was excavated in large chunks making it difficult to form uniform lifts. Once dumped, the fill material was manipulated with a bulldozer. Compaction was achieved through the use of a sheepsfoot roller and/or by tracking the material down with a bulldozer.

On November 10, 1997 boring B-5 was drilled at the referenced location to evaluate the density, water content, and unconfined compressive strength on representative portions of the completed embankment. The results are shown on Boring Log B-5 Appendix B. The boring density tests indicated that the percent compaction ranged from approximately 79 percent to over 100 percent of the standard Proctor with moisture contents ranging from –3.0% to +16.4% from optimum. The boring density tests are shown on Figure 16 with respect to the standard Proctor moisture-density relationship and corresponding zero air-void curve. Many of the tests are approaching the zero air-void curve. Two tests taken just above the foundation at 19.3 and 21.2

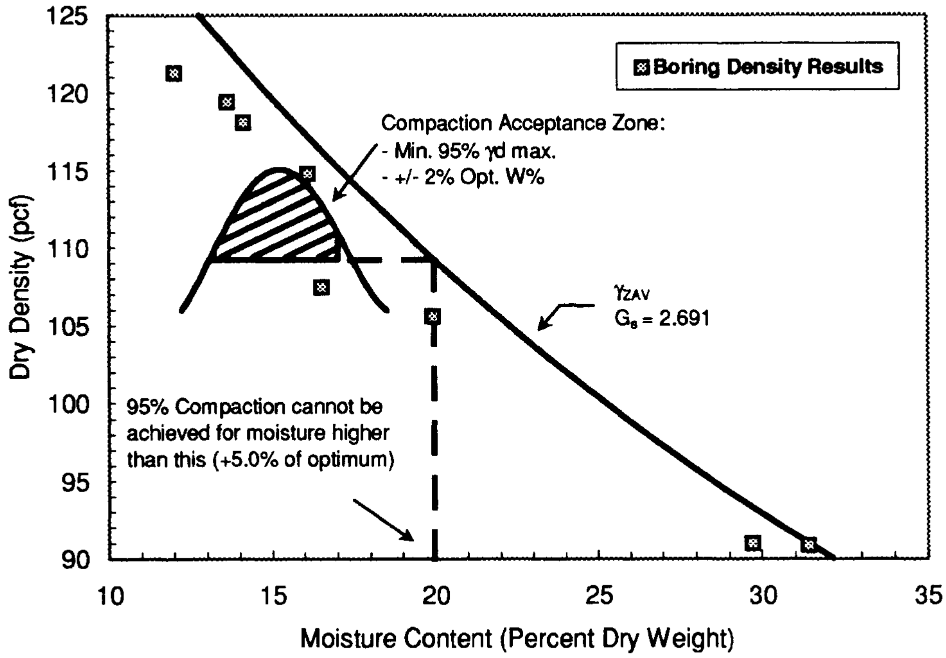


FIGURE 16 US Highway 163 Bypass, Prairie City - Boring density results after embankment construction with corresponding moisture-density relationship and zero air-void curve

feet below the top of embankment had density of 79 percent with moisture contents of +14.7% to +16.4% above optimum. Once again, the materials with high moisture content and low density near the foundation are subject to settlement from consolidation and may set the stage for potential slope failure.

Some of the material in boring B-5 had low shear strength. This would be expected given the high moisture content and low densities of some of the samples. The shear strength of the embankment we plotted against the depth and the factor of safety against reaching the ultimate shear strength of the embankment is shown on Figure 17. Also apparent from this figure is the variability of shear strength throughout the entire embankment, which can lead to differential settlement, consolidation and isolated shear failures. At approximately 5 feet below design subgrade elevation the shear strength is only 1000 lb/ft^2. At 10 feet and 19 feet below the top of embankment the shear strengths are only as high as the minimum needed (FS = 1) to support the foundation's own weight. This embankment again may be prone to potential slope failure.

The soil classification and Atterberg limits of the soil encountered in boring B-5 are shown in Figure 18. Most of the embankment was constructed of soils classified as lean clay. However, at the surface what are supposed to be the select materials appear to be Unsuitables. From 0 to 1 foot below finished subgrade elevation the Liquid Limit was 58 percent with a Plastic Index of 41. This material was misidentified in the field by Iowa DOT personnel and used as Select material and should have been placed below grade according to the current specification. Throughout the embankment the Liquid Limit varied from 33 to 58 percent with respective Plasticity Indices of 16 percent and 41 percent.

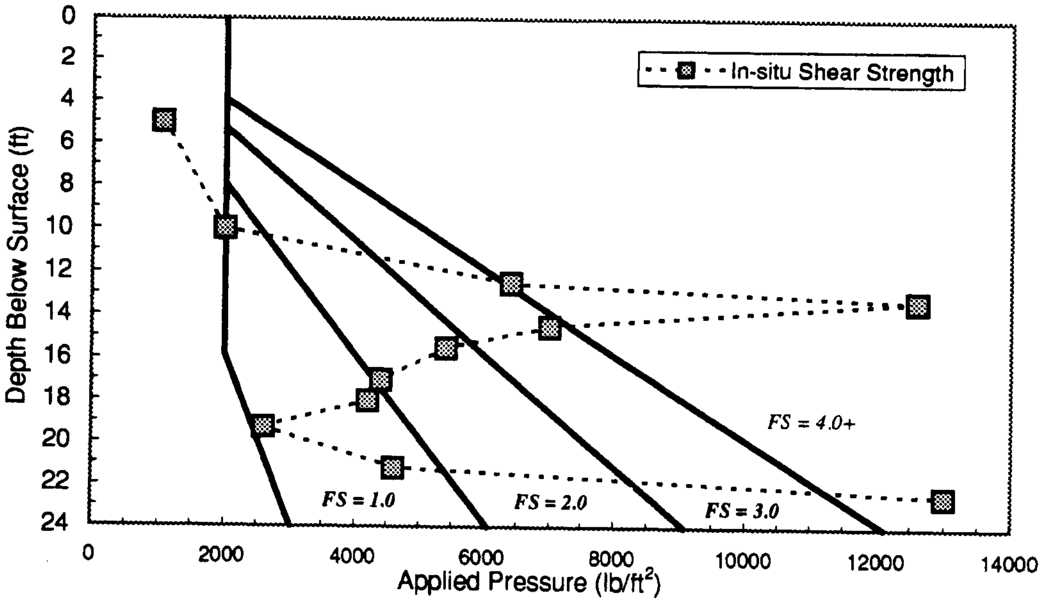
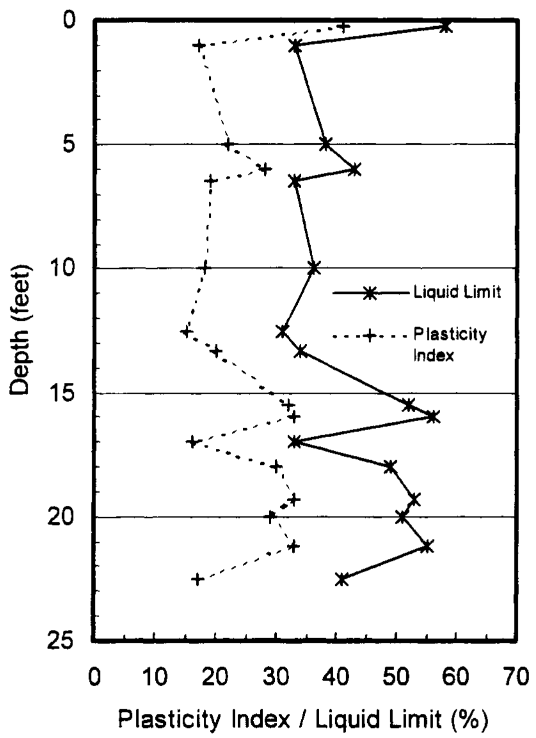


FIGURE 17 US Highway 163 Bypass, Prairie City - The in-situ shear strength at completion of the constructed embankment



Depth	AASHTO Classification	Unified Classification
0-1.0	A-7-6(35)	CH
1.0-3.0	A-6	CL
5.0-5.6	A-6	CL
6.0-6.5	A-7-6(15)	CL
6.5-7.0	A-6	CL
10.0-10.5	A-6	CL
12.5-13.0	A-6	CL
13.3-13.9	A-6	CL
15.5-16.0	A-7-6(33)	CH
17.0-17.5	A-6	CL
18.0-18.6	A-7-6(26)	CL/CH
19.3-19.8	A-7-6(29)	CH
20.0-20.5	A-7-6(32)	CH
21.2-21.8	A-7-6(35)	CH
22.5-23.0	A-7-6(14)	CL

FIGURE 18. Atterberg limits with soil classification profile for boring B-5

Relocation of US Highway 75 - Woodbury County - Sioux City

Between October 30 and 31, 1997 observations and field density tests were performed near station 680+00 at centerline of the east lane of the proposed US Highway 75 relocation. Structural fill material at this location was taken from cut sections and placed in low areas. At this location the soil used as fill material was obtained from loess deposits. Field density tests were performed on fill placed approximately 13 to 18 feet below design subgrade elevation. As shown in Figure 19 the fill material was fairly uniformly compacted with densities ranging from 87 percent to over 100 percent. Moisture contents were on the dry side of optimum and varied from –4.2% to +0.3% from optimum. Field test results indicate that the sheepsfoot walkout specifications appeared to be working reasonably well for this soil type.

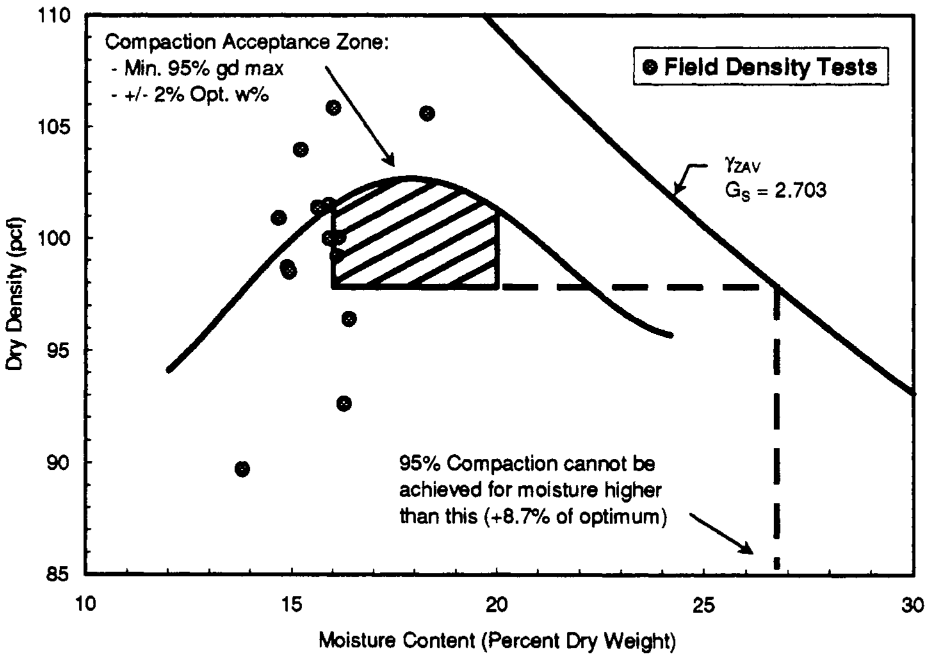


FIGURE 19 US Highway 75 Relocation - Field density results during construction with corresponding moisture-density relationship and zero air-void curve

Summary of Field Investigations

Table 5 summarizes the relative adequacy of the sheepsfoot walkout specification relative to producing a quality embankment for the projects investigated in Phase I.

Out of the six projects, the sheepsfoot specification appeared to produce reasonably good results on only two of the projects where the predominant soil types were lean clays to clayey silt soils.

On three of the projects where the predominant soil types were fat clays the sheepsfoot walkout specification was resulting in embankments being constructed very wet (nearing 100 percent saturation, having low shear strength (stability) and low in-place dry densities. This can be expected to result in differential short and long-term settlements as consolidation slowly takes place and sets the stage for potential slope failures.

On one project, the sheepsfoot walkout was being inappropriately used on a cohesionless material and incorrect test methods were being used to evaluate compaction.

Based on these results, it is our opinion that alternative embankment construction specifications need to be developed, for the various Iowa soil types, in order to construct quality embankments.

TABLE 5 Adequacy of sheepsfoot walkout specifications relative to embankment quality

City	Project	Predominant Embankment Soil Type	Sheepsfoot Walkout Adequacy
1. Mason City	Relocation of US 18–I 35 East to Raven Avenue	Fat Clays to Lean Clays	Inadequate
2. Waverly	Relocation of US 218	Lean Clays	Reasonably Good
3. Sioux City	Relocation of US 75	Clayey Silts	Reasonably Good
4. Monroe	Highway 163 Bypass	Fat Clays	Inadequate
5. Mason City	Relocation of US 18-Raven Ave to Winnebago River	Sands	Not Appropriate
6. Prairie City	Highway 163 Bypass	Fat Clays	Inadequate

RAPID IN-SITU TESTING RESULTS

"Speedy" Calcium Carbide Gas Pressure Tester

The "Speedy" moisture tester determines the water content of soil by chemical reaction using calcium carbide as a reagent to react with water in the soil. The reaction produces a gas that creates pressure in the test chamber. A dial reading indicates the increase in pressure and is correlated to percent moisture by wet weight.

When used in the field, the "Speedy" is a practical tool to get quick moisture content results. The advantage of the "Speedy" is that it can be used by field personnel where the soil was sampled (the material does not have to be transported back to the laboratory). An average test takes from 4 to 7 minutes. The highly plastic clay soils that are not friable enough to break up take longer to test than more friable material such as sandy clays.

Several tests were performed on a wide range of soils. From these tests a calibration curve was developed showing the "Speedy" dial reading versus the oven dry moisture contents as shown in Figure 20. For the calibration curve the "Speedy" dial readings were converted from wet weight to percent by dry weight. In addition to the calibration curve, a relationship between the dial reading and percent moisture by dry weight was produced as shown in Figure 21.

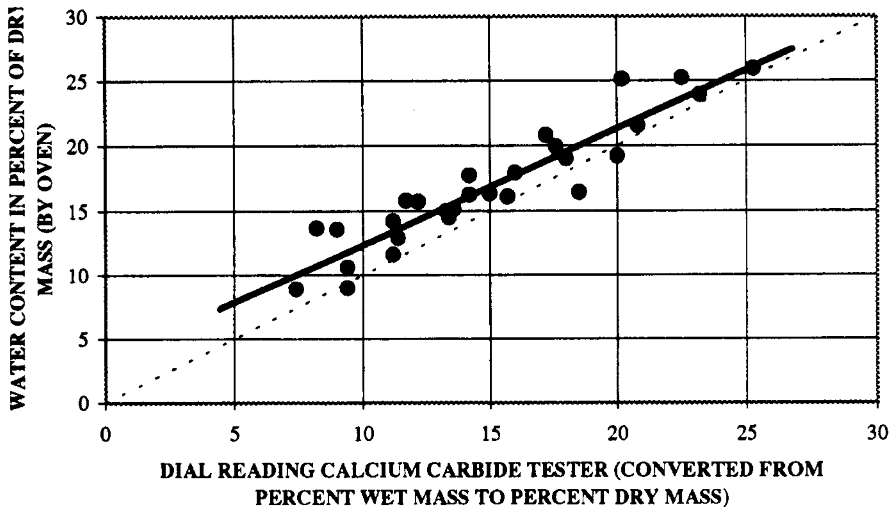


FIGURE 20 Calibration Curve - "Speedy" calcium carbide tester ASTM D-4944

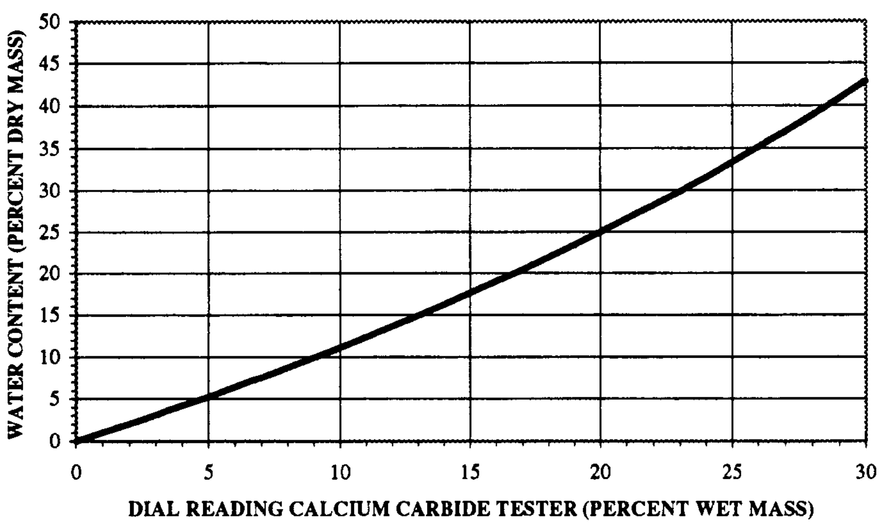


FIGURE 21 "Speedy" calcium carbide dial reading vs. water content by dry mass

It should be noted that when combined with water, the calcium carbide reagent produces a highly flammable or explosive acetylene gas. The test should not be carried out near open flames. As an added precaution, ASTM suggests that the operator use a dust mask, clothing with long sleeves, gloves, and goggles to keep the reagent from irritating the eyes, respiratory system, or skin.

In general the Speedy produced results that were within approximately ±2% of actual moisture content up to approximately 20 percent. It should be noted that extra care must be exercised to select specimens that are representative of the soil. The test is not a foolproof method to determine moisture content and consequently the operator can greatly influence the test results. For example, the specimen may not be agitated enough or prior to taking the dial reading the gas within the chamber has not had adequate time to cool and stabilize. Again the operator must be aware that the dial reading is percent moisture by wet weight. However, most of the standard moisture density relationships use percent by dry weight.

According to ASTM, the precision of this method has not been determined. Data are being evaluated to determine the precision of this test method.

Proctor Penetrometer H-139

The Proctor Penetrometer (developed by the U.S. Army Corp of Engineers) is used to establish the moisture penetration resistance relations of fine grained soil. The soil specimen must have at least 20 percent passing the #200 (75µm) sieve. The Proctor Penetrometer is furnished with needles having end areas of 1, 3/4, 1/2, 1/3, 1/5, 1/10, 1/20, 1/30 and 1/40 square inches. The operator changes the needles so that the needle used will be of such size that the penetrometer reading will be between 20 and 80. For dry stiff soils the 1/30 and 1/40 needles are used and as the moisture content increases the larger needles are used.

Factors also affecting the penetrometer reading include the rate of penetration and the depth of penetration. ASTM D1558 indicates that the penetration rate should be 0.5 inches per second for a distance of not less that three inches.

The penetration resistance versus moisture content are plotted on the following Figure 22 through Figure 27. The plots indicate that the strength of compacted clayey soils generally decreases with the increase in moisture content. Note that at approximately optimum moisture content there is a significant loss of strength. This means that, if two samples are compacted to the same dry unit weight, one of them on the dry side of the optimum and the other on the wet side of the optimum, the specimen compacted on the dry side of the optimum will exhibit greater strength.

Field observations indicated that the use of the Proctor Penetrometer was useful to evaluate the moisture content and suitability of the fill material. On a large section of structural fill being placed on the north bridge berm at the Prairie City Bypass the soil was so wet and unstable that a reading could not be taken. This should immediately inform an operator that the fill section was not stable and too wet.

The disadvantage of this test is that it only tests a small area at the surface of the soil.

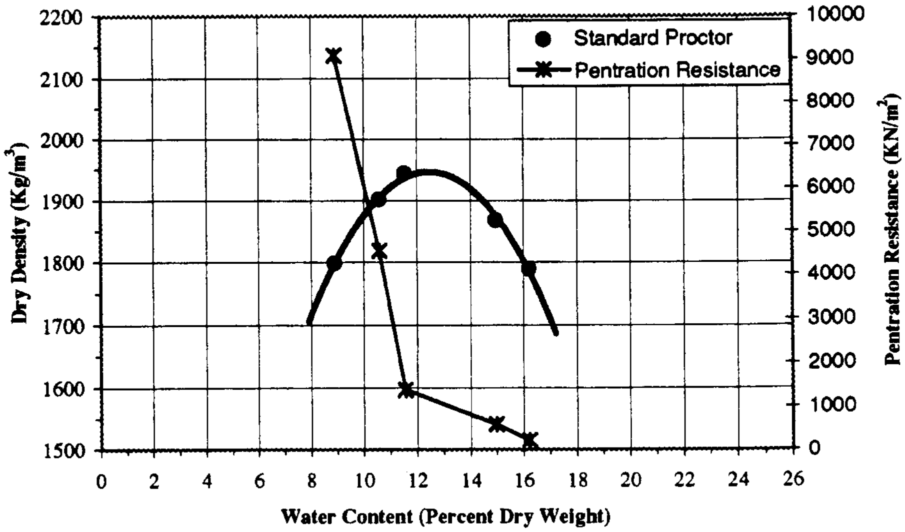


FIGURE 22 Moisture density relationship with corresponding penetration resistance sample no. (1) NHS-18-5(115)-19-17

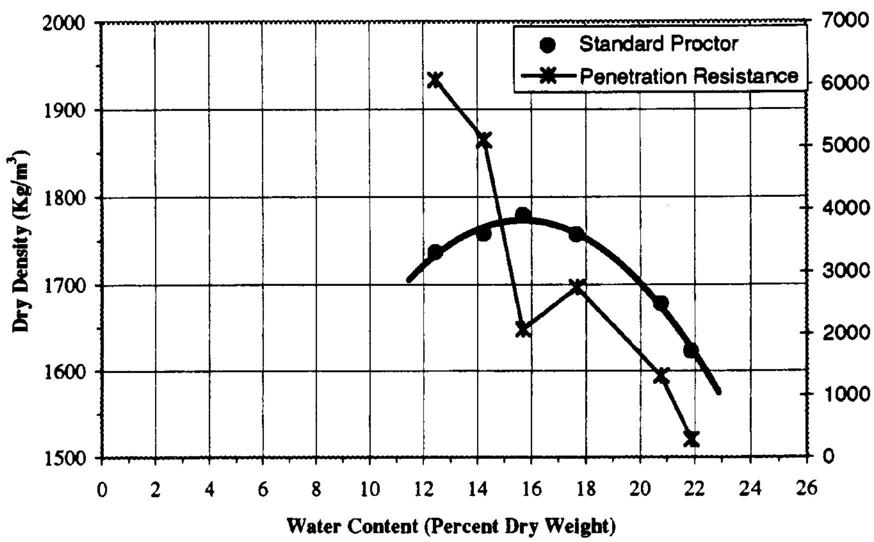


FIGURE 23 Moisture density relationship with corresponding penetration resistance sample no. (1) NHS-218-8(47)-19-07

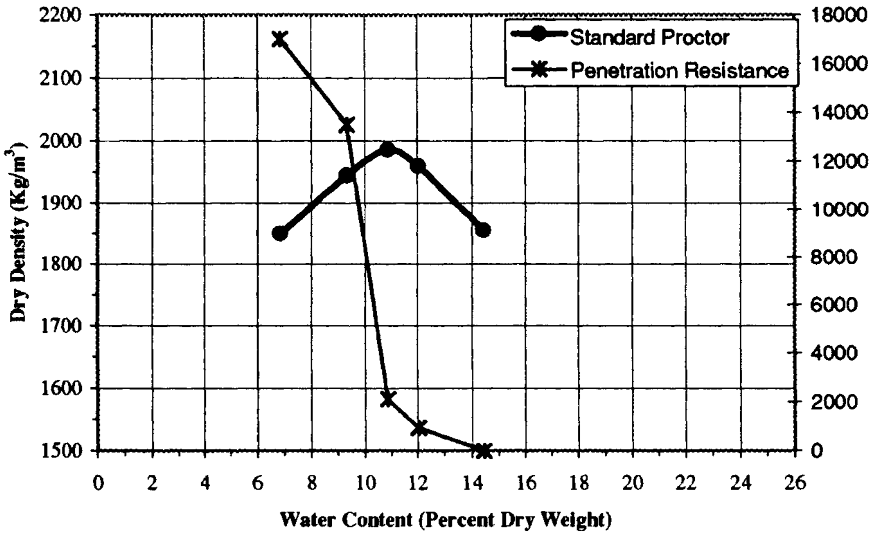


FIGURE 24 Moisture density relationship with corresponding penetration resistance sample no. (1) NHS-18-5(68)-19-17

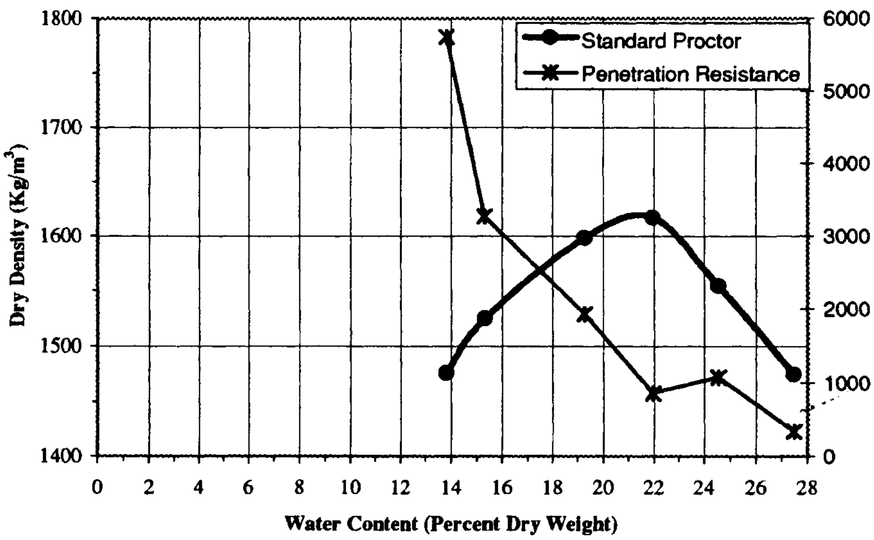


FIGURE 25 Moisture density relationship with corresponding penetration resistance sample no. (1) NHS-18-5(80)-19-17

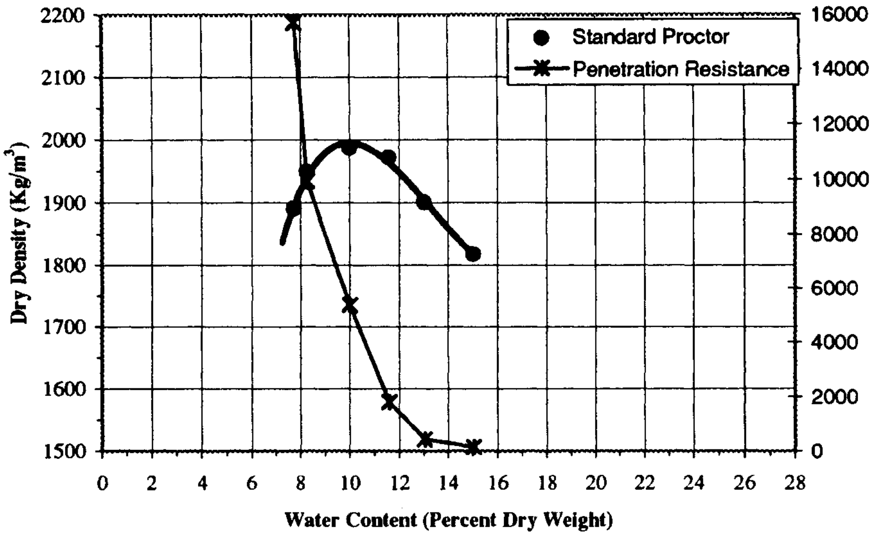


FIGURE 26 Moisture density relationship with corresponding penetration resistance sample no. (1) NHS-18-5(111)-19-17

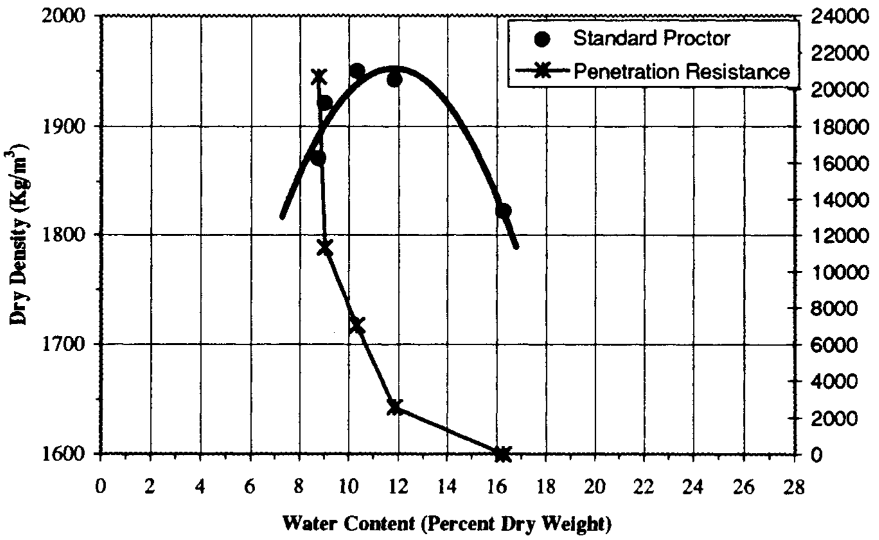


FIGURE 27 Moisture density relationship with corresponding penetration resistance sample no. (1) NHS-18-5(117)-19-17

Ely Volumeter with "Speedy" Moisture

Moisture-density relationships were developed using the Ely volumeter and the "Speedy" calcium carbide moisture tester. In order to correlate these findings with the AASHTO T-99 standard Proctor curves, a Proctor specimen was used as the test sample. The Ely volumeter sample was taken from the compacted proctor sample and the "Speedy" sample was taken from the core of the same sample.

As can be seen from the Figures 28 and 29, the Ely and "Speedy" combination yielded comparable but imprecise results. The Ely and "Speedy" combination curves yielded both greater and lesser densities than the standard Proctor curves.

The variation in the sample size between the standard Proctor and the Ely volumeter is more than likely the cause of the variation between the tests. The foreseeable problem with the Ely volumeter is obtaining a representative sample from the embankment material. The sample is small enough that large variations in densities could be obtained from a relatively small area. The Ely volumeter is a rapid test but the small sample size precludes its use in field applications.

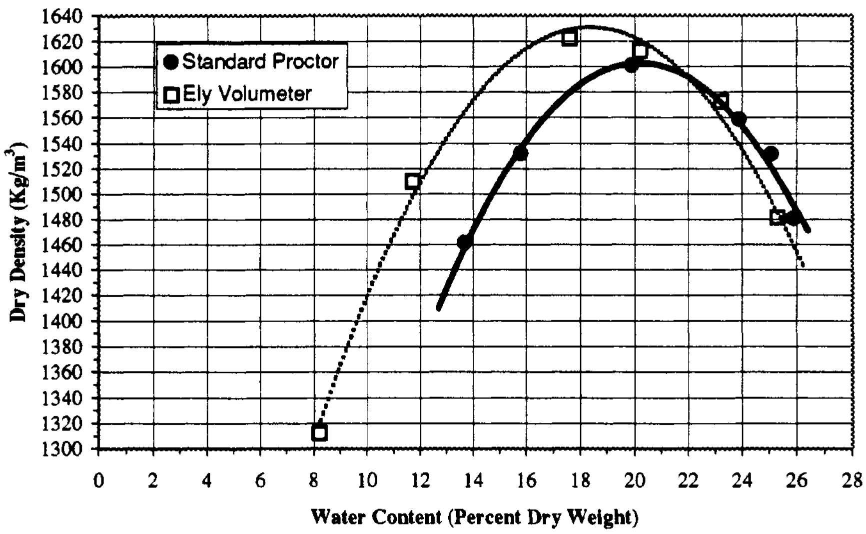


FIGURE 28 Ely volumeter & "Speedy" moisture density relationship compared with standard Proctor curve sample no. (1) NHSN-163-2(13)-2R-50

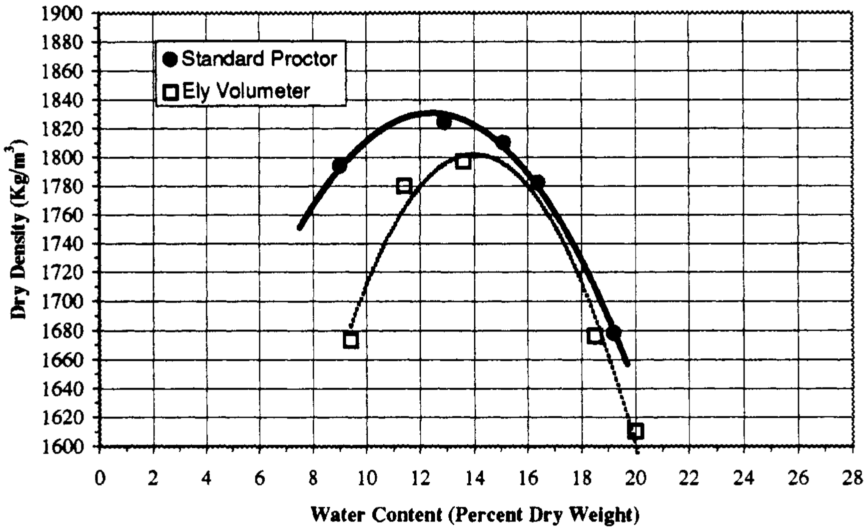


FIGURE 29 Ely volumeter & "Speedy" moisture density relationship compared with standard Proctor curve sample no. (2) NHSN-163-2(13)-2R-50

One-Point Proctor Curve Iowa Test Method No. 103-D

The Iowa 103-D One-Point method for determining optimum moisture and density has been compared to the standard Proctor method ASTM D 698.

The one-point test procedure begins first with calculating the moisture content and wet unit weight of a one-point specimen. Then the point of intersection of the wet unit weight and moisture content is plotted in the family of curves entitled "Moisture Density Curves" currently in use by the Iowa DOT. If the plotted point falls outside the "Range of Confidence," then another specimen is to be recompacted that will place the point within this range.

The one-point results are shown on the Figures 30 through Figures 34, along with the standard Proctor. The one-point results are from approximately -5% to +3% within the optimum moisture and density of the standard Proctor values. Some of the one-point test points plotted on the following figures were not within the "Range of Highest Confidence"; however, the points were plotted to show that large deviations exist when outside this range.

The One-Point Method appears to be a reasonable method to determine the optimum moisture-density relationship for a rough estimate, although the standard Proctor relationship would provide more accurate results for use with acceptance testing.

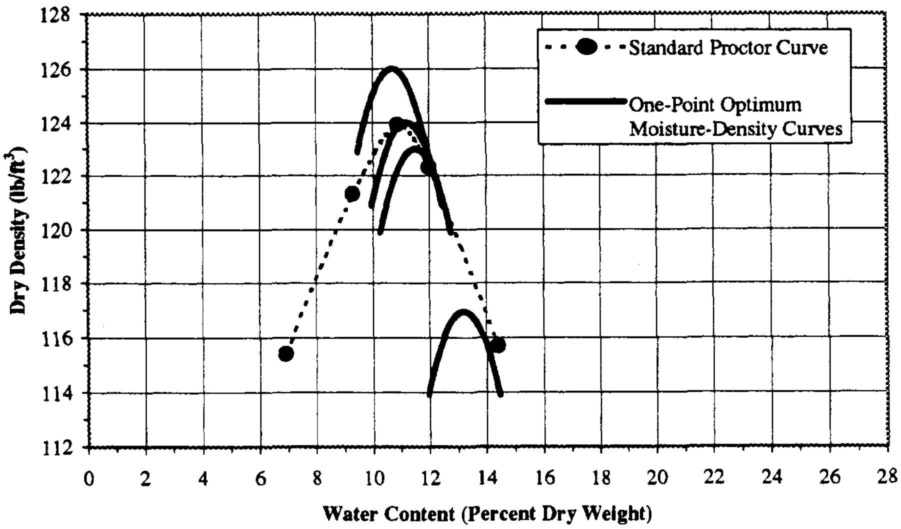


FIGURE 30 Comparison of one-point optimum moisture-density curves by test method Iowa 103-D - sample no. (1) - NHS-218-8(47)-19-07

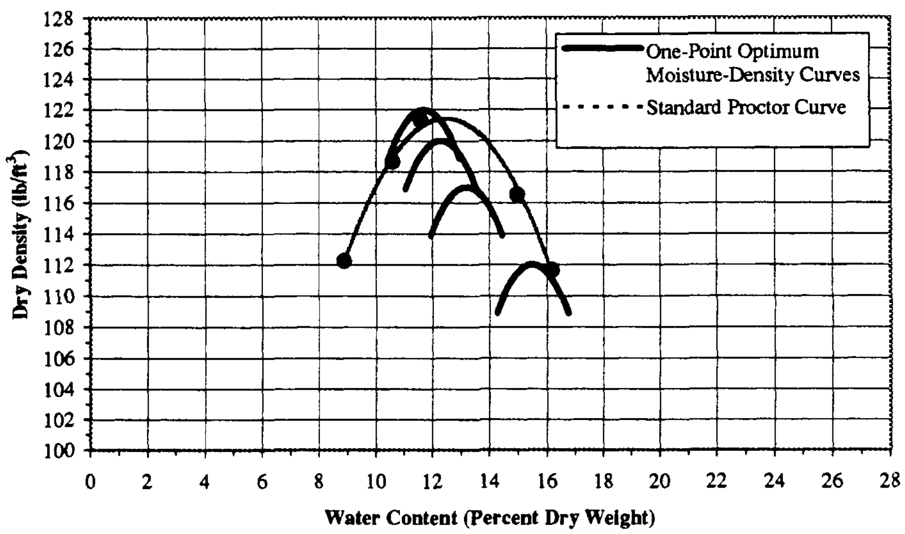


FIGURE 31 Comparison of one-point optimum moisture-density curves by test method Iowa 103-D - sample no. (1) - NHS-18-5(115)-19-07

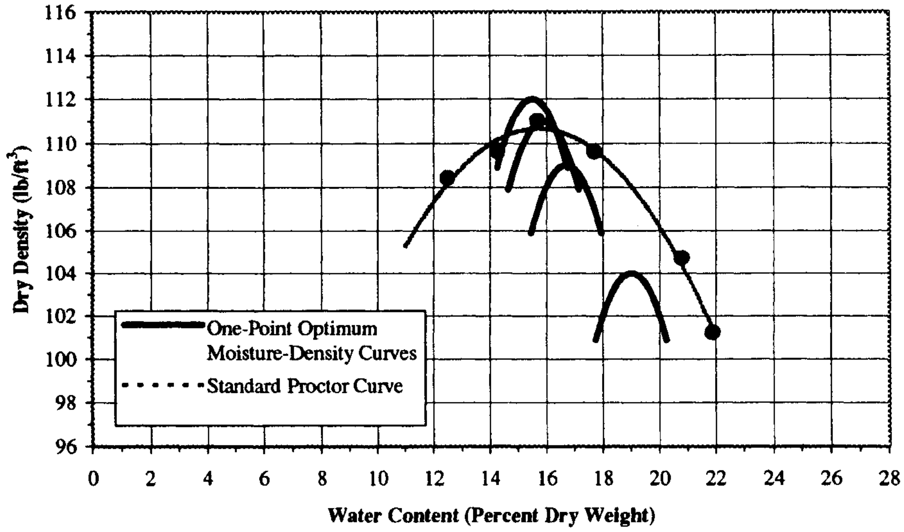


FIGURE 32 Comparison of one-point optimum moisture-density curves by test method Iowa 103-D - sample no. (2) - NHS-218-8(47)-19-07

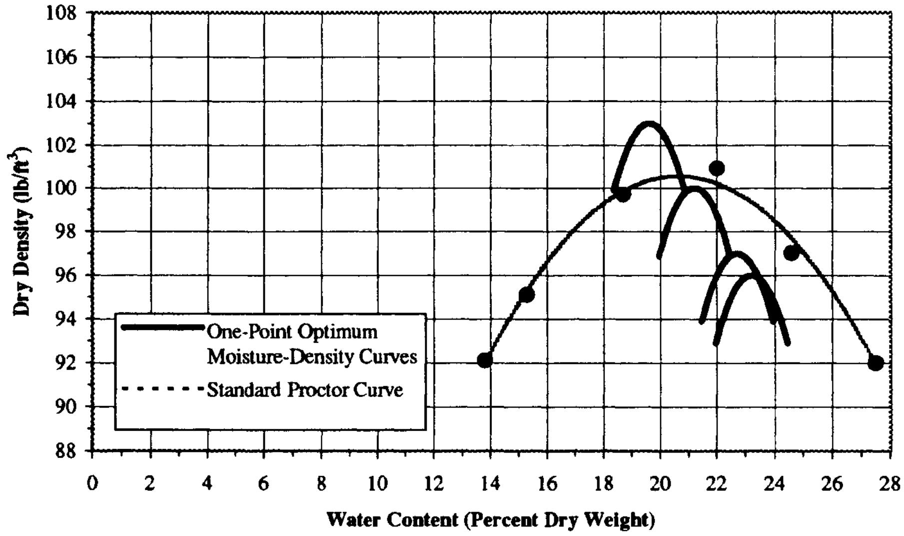


FIGURE 33 Comparison of one-point optimum moisture-density curves by test method Iowa 103-D - sample no. (1) - NHS-18-5(80)-19-17

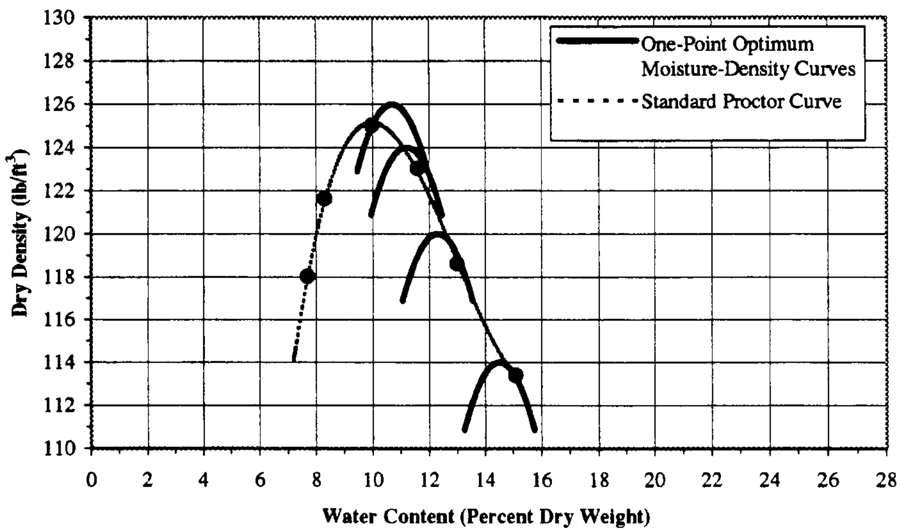


FIGURE 34 Comparison of one-point optimum moisture-density curves by test method Iowa 103-D - sample no. (1) - NHS-18-5(111)-19-17

These results indicate that for the one-point Proctor method to be accurate, the moisture content must be close to the "optimum" moisture content of the soil. Since this is unknown in the field this makes it difficult for field personnel to interpret test data.

SUMMARY OF IN-SITU TESTING METHODS

During the field investigations and testing, several types of available equipment were utilized to determine the in-place moisture content, density, and strength of embankment fill materials

"Speedy" Calcium Carbide Gas Pressure Tester

The "Speedy" moisture tester determines the water content of soil by chemical reaction using calcium carbide as a reagent to react with water in the soil. The reaction produces a gas that creates pressure in the test chamber. A dial reading indicates the increase in pressure and is correlated to percent moisture by wet weight. When used in the field, the "Speedy" was a practical tool to get quick moisture content results. The advantage of the "Speedy" is that it can be used by field personnel where the soil was sampled (the material does not have to be transported back to the laboratory). An average test takes from 4 to 7 minutes. In general the speedy produced results that were within approximately ±2% of actual moisture content and would be a good tool for field control.

Proctor Penetrometer H-139

The Proctor penetrometer was used to establish the moisture penetration resistance relations of fine-grained soil. The soil specimen must have at least 20% passing the #200 (75μm) sieve according to ASTM Test Designation D-1558. The Proctor penetrometer is furnished with needles having end areas of 1, 3/4, 1/2, 1/3, 1/5, 1/10, 1/20, 1/30 and 1/40 square inches. The operator changes the needles so that the needle used will be of such size that the penetrometer reading will be between 20 and 80.

A typical plot indicates that the strength of compacted clayey soils generally decreases with the increase in moisture content. Field observations indicated that the use of the Proctor penetrometer was useful to evaluate the moisture content and stability of the fill material; however, it only is used on the surface of the soil and tests only a small area.

Ely Volumeter with "Speedy" Moisture

The Ely volumeter is a small hand held apparatus that is pushed into the soil and extruded. The soil specimen is trimmed to a known volume and then weighed to determine the density. Moisture-density relationships were developed using the Ely volumeter and the "Speedy" calcium carbide moisture tester. In order to correlate these findings with the AASHTO T-99 standard Proctor curves, the proctor specimens were used as the test sample. The Ely and "Speedy" combination yielded comparable results with the standard Proctor. In general the Ely and "Speedy" combination is a quick and fairly accurate in-situ test to obtain density and moisture; however, the sample size of the Ely limits its use in practical applications.

Army Corps of Engineers Surface Soil Sampler

The Corps of Engineers Surface Soil Sampler was developed to take density or compaction tests at or near the ground surface. The density sampler consists of a 10-lb. drop hammer and thin walled steel tubes machined to a calibrated volume. The steel tubes were driven into the ground then removed, trimmed and weighed to obtain wet density. A moisture content sample was then obtained from the center of each tube. Two sizes of tubes are available; however, the 4" O.D. × 4" long were used through our testing. These tests were performed in accordance with ASTM Test Designation D-2937. This appears to be a usable field test.

Liquid Limit Tests

The device used in this test consists of a brass cup that is dropped onto a base plate by cranking the cam. Soil is placed in the brass cup and a groove is placed in the center. The moisture content required to close the groove 0.5 inches at 25 blows is defined as the liquid limit. Generally several tests are performed for a given soil and the results plotted on a log scale to determine the liquid limit. However, a one-point method is available for quicker results and could be utilized for a quick field tool.

Plastic Limit Tests

The plastic limit is defined as the moisture content at which the soil will crumble, when rolled into a thread of 1/8 inch in diameter. The equipment used in this test consists of a smooth rolling plate, which is typically glass, and a source of water.

Dynamic Cone Penetration

For future research the Dynamic Cone Penetration (DCP) test will be utilized to measure the strength/stability of embankment fill sections. Furthermore, the in-situ soil thickness and layers can be identified. Typically, the DCP can penetrate from 0 to 5 feet. The DCP device consists of two thin diameter steel shafts coupled at the middle. The lower shaft contains an anvil and pointed tip. The strength of the soil is measured by counting the number of blows for a measured penetration. Because the failure mechanisms are similar the DCP and CBR test results have been correlated with one another. Therefore, with the use of the DCP and CBR correlations, a project can be designed for site-specific results.

Recently, the Minnesota, Illinois, and Kansas Departments of Transportation are investigating the DCP for testing cohesive and cohesionless soils for use in road construction. This method is advantageous because it is fast, accurate, versatile, economical, and easy to use. It also provides a means of evaluating density and stability (strength) at the same time.

PHASE I RESULTS

The results of Phase I field and laboratory construction testing and observations and post construction testing are summarized as follows.

Field Personnel (Iowa DOT and Contractors) Observations

Appear to be generally conscientious and trying to do a good job, but:

- Are misidentifying soils ("unsuitable" and "Class 10" soils being used as "select").
- Lack soil identification skills (knowledge and equipment).
- Are relying heavily on soil design plan sheets for determining unsuitable, suitable and select soils (soils design data appear accurate, and are necessary, but spacing between borings and soil mixing during construction operations makes it difficult to differentiate the soils in the field).

Current Specifications

- Method of identifying unsuitable, suitable, and select soils may not be adequate.
- One point Proctor does not appear adequate for identifying all soils or for field verifications of compaction for all soils.
- "Sheepsfoot walkout" is not, for all soils, a reliable indicator of
  - → Degree of compaction
  - → Compaction moisture content
  - → Adequate stability

Construction Observations and Testing – Cohesive Soils

- Sheepsfoot walkout specification producing embankments where
  - → Soils are being placed wet of standard Proctor optimum moisture.
  - → Soils are being compacted to near 100% saturation resulting in an embankment that:
    - ▸ Has low soils shear strength (inadequate stability).
    - ▸ Has a possibility of positive pore water pressure development (as embankment height increases) which results in a further reduction in shear strength.
    - ▸ Sets the stage for potential slope failure.
- Discing and lift leveling specification not always being enforced (particularly important for disposal of unsuitables and compaction of "fat" clayey soils).
- Lifts being placed on overcompacted soils – overcompaction was evidenced by rutting under loaded scrapers and/or truck haul units.

Construction Observations and Testing – Cohesionless Soils

- Compaction being attempted with sheepsfoot rollers (vibratory necessary).
- Compaction being monitored using standard Proctor testing which is an inappropriate method and can grossly overestimate degree of compaction.

Overall evaluation of the results of Phase I indicate that we are not consistently obtaining a quality embankment constructed under the current Iowa DOT specifications.

PHASE I RECOMMENDATIONS

1. Develop training programs and workshops for field personnel (Iowa DOT and Contractors) for:
   A. Soil identification and classification
   B. Soil compaction basics
      - Cohesive soils
      - Intergrade soils
      - Cohesionless soils

2. Investigate, develop and provide field soils testing kits for
   A. Rapid identification and classification of soils
   B. Rapid determination of soil moisture and density/stability characteristics
      - Cohesive soils (proctor method) in combination with dynamic cone penetrometer (DCP)
      - Cohesionless soils (relative density) in combination with DCP

3. Investigate replacement of part of current methods for identifying "unsuitable," "suitable" and "select" soils.
   A. Use Atterberg Limits (PI) and percent passing #200 sieve.
   B. Use Unified Classification in addition to AASHTO.
   C. Retain percent Carbon specification.

4. Investigate replacement of sheepsfoot walkout specification for cohesive soils.
   A. Use control strips, Proctor density and DCP density/stability testing to establish field rolling patterns and moisture range requirements for each major soil change.
      - Establish minimum DCP from base to 5 feet from top.
      - Establish minimum DCP from 5 feet level to top.
   B. Use spot DCP density/stability testing to check compaction on the portion of the embankment less than 5 feet.
   C. Use lift testing for top 5 feet.

5. Require vibratory compaction for cohesionless soils.
   A. Moisture control – contractor option.
   B. Minimum 90% relative density required correlated with DCP.
   C. Use test strips to establish rolling pattern, equipment operation, and minimum DCP.
   D. Spot-check with DCP for relative density requirement.

ACKNOWLEDGMENTS

The authors gratefully acknowledge and thank Iowa DOT technical advisor John Vu for his cooperation, timely assistance, and advice throughout Phase I.

We also thank the steering committee Bob Stanley, Dave Woofter, Bob Jimerson, Jerry Danforth and John Moyna for their advice and counsel. We are especially grateful for the field input received from the Iowa DOT construction office personnel.

A special thanks to the Iowa DOT drilling crews, and to Bob Stanley for his cooperation and willingness to help in conducting and coordinating post construction drilling and sampling.

REFERENCES

1. Selig, E.T. "Variability of Compacted Soils." National Conference on Statistical Quality Control Methodology in Highway and Airfield Construction, Nov. 1966, pp. 181-213.

2. U.S. Army Engineer Waterways Experimentation Station, "Soil Compaction Investigation." Vicksburg MS, *Technical Memorandum 3-271*, (1949) , Reports 2 and 3.

3. Livneh, M., G. Kassif, and G. Wiseman. "The Use of Index Properties in the Design of Pavements on Expansive Clays." *Proceedings, Second Conference on Expansive and Collapsing Soil*, Texas A & M Univ., College Station, 1969. pp. 218-234.

4. Hilf, J.W. "A Rapid Method of Construction Control for Embankments of Cohesive Soil." *Engineering Monograph No. 26*, Bureau of Reclamation, (1959). p. 7-23.

5. Rollings, Marian P. and Raymond S. Rollings Jr. *Geotechnical Materials in Construction.* McGraw Hill, New York, (1996). pp. 119-389.

6. Kosmatka, Steven H. and Panarese, William C., *Design and Control of Concrete Mixtures*, Thirteenth Edition, Portland Cement Association, 1988.

7. Schmidt, Norbert, O. and Charles O. Riggs, "Methods for Achieving and Measuring Soil Compaction", *Concrete Construction.* (1995) pp. 681-689.

8. Terzaghi, Karl. *From Theory to Practice in Soil Mechanics*, Wiley and Sons, 1960.

Appendix A

Civil Engineering Department
Iowa State University
Geotechnical Laboratory

FIELD DENSITY TEST

PROJECT: Mason City		Nuclear Density Gage	ND
SITE: Cerro Gordo County	HOLE	Army Corps Sampler	AS
SAMPLE	DEPTH: -20 to -10 ft	Shelby Tube	ST
LOCATION: Sta. 2176+00 to 2178+00			
TECHNICIAN: DW/MW	DATE: 8/5/97-10/19/97		

Test No.	Date	Location	Lift Elevation	Material Mark	Optimum Density	Optimum Moisture	Dry Density	Percent Moisture	Percent Compaction	+/- Optimum Moisture	Test Method
1	8/5/97	2176+75 N. Cl 8m	-20ft	A	124.5	11.0	124.0	12.8	100	1.8	ST
2	8/5/97	2176+50 N. Cl 8m	-20ft	A	124.5	11.0	120.1	11.7	96	0.7	ST
3	10/2/97	2176+50 N. Cl 12m	-12ft	B	126.5	10.0	123.9	10.4	98	0.4	AC
4	10/2/97	2176+50 N. Cl 30m	-12ft	B	126.5	10.0	127.2	10.6	100+	0.6	AC
5	10/2/97	2176+75 N. Cl 12m	-12ft	B	126.5	10.0	118.4	14.7	94	4.7	AC
6	10/2/97	2176+75 N. Cl 30m	-12ft	B	126.5	10.0	118.8	10.4	94	0.4	AC
7	10/2/97	2177+00 N. Cl 12m	-12ft	B	126.5	10.0	127.4	11.5	100+	1.5	AC
8	10/2/97	2177+00 N. Cl 30m	-12ft	B	126.5	10.0	128.2	11.2	100+	1.2	AC
9	10/2/97	2177+25 N. Cl 12m	-12ft	B	126.5	10.0	122.7	13.0	97	3.0	AC
10	10/2/97	2177+25 N. Cl 30m	-12ft	B	126.5	10.0	119.6	14.3	95	4.3	AC
11	10/2/97	2177+50 N. Cl 12m	-12ft	B	126.5	10.0	118.0	12.2	93	2.2	AC
12	10/2/97	2177+50 N. Cl 30m	-12ft	B	126.5	10.0	121.5	13.7	96	3.7	AC
13	10/2/97	2177+75 N. Cl 30m	-12ft	B	126.5	10.0	120.4	12.6	95	2.6	AC
14	10/19/97	2176+25 N. Cl 10m	-10ft	C	125.5	10.5	112.8	14.9	90	4.4	AC
15	10/19/97	2176+50 N. Cl 10m	-10ft	C	125.5	10.5	112.2	15.5	89	5.0	AC
16	10/19/97	2176+50 N. Cl 20m	-10ft	C	125.5	10.5	120.8	14.2	96	3.7	AC
17	10/19/97	2176+50 N. Cl 30m	-10ft	C	125.5	10.5	123.3	13.1	98	2.6	AC

Civil Engineering Department Iowa State University Geotechnical Laboratory **FIELD DENSITY TEST**	PROJECT: Monroe Bypass HWY 163 SITE: Jasper County HOLE SAMPLE: DEPTH: -17 to -5 LOCATION: Sta.264+00 to 266+25 Near Center Line TECHNICIAN: DW/MW DATE: 7/29/97- 9/29/97	Nuclear Density Gage ND Army Corps Sampler AC Shelby Tube ST

Test No.	Date	Location	Lift Elevation	Material Mark	Optimum Density	Optimum Moisture	Dry Density	Percent Moisture	Percent Compaction	+/- Optimum Moisture	Test Method
1	7/29/97	264+00 S.CL 1.0 m	-17 ft	A	106.0	19.0	93.6	28.2	88	9.2	ST
2	7/29/97	264+70 S.CL 6.5 m	-17 ft	A	106.0	19.0	94.9	25.2	90	6.2	ST
3	7/29/97	264+55 S.CL 23.0 m	-17 ft	A	106.0	19.0	101.5	22.7	96	3.7	ST
4	7/29/97	264+30S.CL 1.0 m	-17 ft	A	106.0	19.0	97.2	22.0	92	3.0	ST
5	7/29/97	264+20 S.CL 18.5 m	-16 ft	A	106.0	19.0	97.0	26.6	92	7.6	ST
6	8/6/97	264+50 S.CL 10.0 m	-16 ft	B	106.0	19.0	102.6	20.1	97	1.1	ST
7	8/6/97	264+75 S.CL 9.5 m	-15 ft	B	106.0	19.0	99.1	25.0	93	6.0	ST
8	8/6/97	265+00 S.CL 16.0 m	-14 ft	B	106.0	19.0	99.7	23.0	94	4.0	ST
9	9/6/97	264+00 S.CL 10.0 m	-11 ft	C	102.0	18.0	87.4	25.9	86	7.9	ND
10	9/6/97	264+50 S.CL 10.0 m	-11 ft	C	102.0	18.0	98.3	25.8	96	7.8	ND
11	9/6/97	264+75 S.CL 10.0 m	-11 ft	C	102.0	18.0	99.3	24.1	97	6.1	ND
12	9/6/97	265+00 S.CL 10.0 m	-11 ft	C	102.0	18.0	96.3	23.8	94	5.8	ND
13	9/6/97	265+10 S.CL 10.0 m	-11 ft	C	102.0	18.0	101.1	19.9	99	1.9	ND
14	9/6/97	265+25 S.CL 10.0 m	-11 ft	C	102.0	18.0	99.8	21.9	98	3.9	ND
15	9/6/97	265+50 S.CL 15.0 m	-11 ft	C	102.0	18.0	92.3	23	90	5.0	ND
16	9/6/97	265+60 S.CL 15.0 m	-11 ft	C	102.0	18.0	89.8	24.5	88	6.5	ND
17	9/29/97	265+25 S.CL	-5 ft	D	105.5	18.5	89.0	30.6	84	12.1	AC
18	9/29/97	265+50 S.CL	-5 ft	D	105.5	18.5	103.9	22.3	98	3.8	AC
19	9/29/97	266+25 S.CL	-5 ft	D	105.5	18.5	92.1	27.9	87	9.4	AC
20	9/29/97	264+75 S.CL	-5 ft	D	105.5	18.5	96.8	26.6	92	8.1	AC
21	9/29/97	266+00 S.CL 10.0 m	-5 ft	D	105.5	18.5	105.8	21.5	100	3.0	AC

Civil Engineering Department
Iowa State University
Geotechnical Laboratory

FIELD DENSITY TEST

PROJECT: Sioux City
SITE: Relocation Hwy 65 HOLE
SAMPLE: "A" DEPTH: -14ft to -18ft
LOCATION: Sta 677+00 to 680+00
TECHNICIAN: DW/MW DATE: 11-4-97

Nuclear Density Gage	ND
Army Corps Sampler	AC
Shelby Tube	ST

Test No.	Date	Location	Lift Elevation	Material Mark	Optimum Density	Optimum Moisture	Dry Density	Percent Moisture	Percent Compaction	+/- Optimum Moisture	Test Method
1	10/30/97	Sta. 680+00	-13	A	103.0	18.0	100.9	14.7	98	-3.3	AC
2	10/30/97	Sta. 679+20	-14	A	103.0	18.0	101.4	15.6	98	-2.4	AC
3	10/30/97	Sta. 678+80	-15	A	103.0	18.0	105.6	16	100+	-2.0	AC
4	10/30/97	Sta. 678+40	-16	A	103.0	18.0	104.0	15.2	100+	-2.8	AC
5	10/30/97	Sta. 677+60	-17	A	103.0	18.0	92.6	16.3	90	-1.7	AC
6	10/30/97	Sta. 677+30	-17	A	103.0	18.0	98.5	14.9	96	-3.1	AC
7	10/30/97	Sta. 677+00	-17	A	103.0	18.0	100.1	16.1	97	-1.9	AC
8	10/31/97	Sta. 680+00	-14	A	103.0	18.0	99.2	16.1	96	-1.9	AC
9	10/31/97	Sta. 679+20	-15	A	103.0	18.0	96.4	16.4	94	-1.6	AC
10	10/31/97	Sta. 678+80	-16	A	103.0	18.0	101.5	15.9	99	-2.1	AC
11	10/31/97	Sta. 678+40	-17	A	103.0	18.0	89.7	13.8	87	-4.2	AC
12	10/31/97	Sta. 677+60	-18	A	103.0	18.0	98.7	14.9	96	-3.1	AC
13	10/31/97	Sta. 677+30	-18	A	103.0	18.0	100.0	15.9	97	-2.1	AC
14	10/31/97	Sta. 677+00	-18	A	103.0	18.0	105.6	18.3	100+	0.3	AC

Civil Engineering Department
Iowa State University
Geotechnical Laboratory

FIELD DENSITY TEST

PROJECT: Waverly Bypass
SITE: Bremer County HOLE
SAMPLE: DEPTH: -18 to -20
LOCATION: Sta. 479+00 to 480+25 Near Center Line
TECHNICIAN: DW/MW DATE: 8/14/97

Nuclear Density Gage	ND
Army Corps Sampler	AC
Shelby Tube	ST

Test No.	Date	Location	Lift Elevation	Material Mark	Optimum Density	Optimum Moisture	Dry Density	Percent Moisture	Percent Compaction	+/- Optimum Moisture	Test Method
1	8/14/97	479+00 C_L	-20 ft	A	119.0	12.5	123.6	10.4	100+	-2.1	ST
2	8/14/97	479+25 C_L	-20 ft	A	119.0	12.5	121.0	13.2	100+	0.7	ST
3	8/14/97	479+50 C_L	-20 ft	A	119.0	12.5	116.1	12.7	98	0.2	ST
4	8/14/97	479+75 C_L	-20 ft	A	119.0	12.5	115.1	16.9	97	4.4	ST
5	8/14/97	480+00 C_L	-18 ft	A	119.0	12.5	116.5	15.5	98	3.0	ST
6	8/14/97	480+25 C_L	-18 ft	A	119.0	12.5	116.6	14.8	98	2.3	ST

Appendix B

Civil and Construction Engineering Department - Iowa State University

LOG OF TEST BORING

Boring Number B-1   Sheet 1 of 2   **Lab Number** 1

Date Drilled	**11/18/97**	Project	**Relocation of US Highway 18**
Surface Elevation			**Cerro Gordo County - Mason City**
Depth Drilled	**24.1 feet**	Client	**Iowa Department of Transportation**
Drilling Method	**Hollow-Stem Auger**		

Depth to Water _______ ft @ completion(∇), _______ ft @ _____ hrs. (▼), _______ ft @ _____ hrs. (▼)

Depth, feet	Samples: Number	Samples: Type	SPT Blows per 6 inches	Moisture Content, %	Dry Density, pcf	Unconfined Compressive Strength, psf	Water Level Depth, feet	Graphic Log	SOIL DESCRIPTION
0									Dark gray clayey sand
									Unsuitable gray brown, silty clay, moist, medium stiff
	1	ST		12.0	121	2110			
5									
	2					*7500			Unsuitable gray brown, silty clay , moist, stiff
	3	ST				*5000			Unsuitable gray brown, silty clay
	4	ST		12.6	109	7640			
10									
	5					*4500			
	6	ST		13.2	120	3890			
									Dark gray clayey sand with silt
15									
	7	ST		15.9	118	4650			Gray clayey sand with silt
	8			12.1	128	9260			
	9			14.8	110	2720			Gray silty sand with clay
	10	ST		11.2	118	2700			Unsuitable dark gray, silty clay, soft to medium stiff
20									
	11	ST		29.5	91	9260			
	12	ST		25.0	100	1200			Poorly graded sand, very moist, low strength
	13			14.4	117	*2000			
									<<B-1>> Drilled to 23.1 feet, sampled to 24.1 feet * Calibrated Penetrometer

TEST BORING IOWA ST.GPJ IOWA ST.GDT 4/8/98

Civil and Construction Engineering Department - Iowa State University

LOG OF TEST BORING

Boring Number B-2 | Sheet 1 of 1 | **Lab Number** 2

Date Drilled	**11/19/97**	Project	**Relocation of US Highway 18**
Surface Elevation			**Cerro Gordo County - Mason City**
Depth Drilled	**24.0 feet**	Client	**Iowa Department of Transportation**
Drilling Method	**Hollow-Stem Auger**		

Depth to Water ______ ft @ completion(▽), ______ ft @ ______ hrs. (▼), ______ ft @ ______ hrs. (▽)

Depth, feet	SAMPLES Number	SAMPLES Type	SPT Blows per 6 inches	Moisture Content, %	Dry Density, pcf	Unconfined Compressive Strength, psf	Water Level Depth, feet	Graphic Log	SOIL DESCRIPTION
0									Well graded sand with gravel ("Red sand")
	1	SS	13/16/19						
	2	SS	25/22/23						
5									
	3	SS	22/11/12						
	4	SS	9/9/12						
10									Black/Brown clay with organics, moist, medium stiff
	5	SS	4/5/8						
									Poorly graded sand with trace gravel ("Light tan sand")
	6	SS	5/8/8						
15									
	7	SS	6/9/11						
	8	SS	6/7/10						
20									Tan clay - Natural foundation material
	9	SS	6/7/11						
	10	ST		DEFT	DEFT	DEFT			

<<B-2>> Drilled to 22.5 feet, sampled to 24.0 feet
DEFT - deformed sample

Civil and Construction Engineering Department - Iowa State University

LOG OF TEST BORING

Boring Number B-3 | Sheet 1 of 1 | **Lab Number** 3

Date Drilled	**11/17/98**	Project	**US Highway 163 Bypass**
Surface Elevation			**Jasper County - Monroe**
Depth Drilled	**20.7 feet**	Client	**Iowa Department of Transportation**
Drilling Method	**Hollow-Stem Auger**		

Depth to Water ______ ft @ completion(▽), ______ ft @ ______ hrs. (▼), ______ ft @ ______ hrs. (▽)

Depth, feet	SAMPLES Number	SAMPLES Type	SPT Blows per 6 inches	Moisture Content, %	Dry Density, pcf	Unconfined Compressive Strength, psf	Water Level Depth, feet	Graphic Log	SOIL DESCRIPTION
0									Unsuitable, Tan high plasticity clay, very moist , stiff consistency
	1	ST		26.0	97	4340			
	2	ST		26.7	86	7500			
	3					*5000			
5									
	4	ST		25.6	87	4470			
10		ST		NO REC		NO REC			
	5	ST		24.4	98	6000			
	6			20.4	105	7760			
15	7	ST				*9000			Class 10, tan lean clay, moist, stiff consistency
	8					*2500			Unsuitable, tan high plasticity clay, moist, stiff
	9	ST		25.9	100	5980			
20	10	ST				*3000			Dark brown lean to fat clay Natural foundation soils

<<B-3>>Boring drilled to 20.2 feet, sampled to 20.7 feet at auger refusal.
* Calibrated Penetrometer
NO REC - No recovery in the field.

Civil and Construction Engineering Department - Iowa State University

LOG OF TEST BORING

Boring Number B-4 Sheet 1 of 2 **Lab Number** 4

Date Drilled	11/20/97	Project	Relocation of US Highway 218
Surface Elevation			Bremer County - Waverly
Depth Drilled	19.2 feet	Client	Iowa Department of Transportation
Drilling Method	Hollow-Stem Auger		

Depth to Water ______ ft @ completion(▽), ______ ft @ ______ hrs. (▼), ______ ft @ ______ hrs. (▼)

Depth, feet	SAMPLES Number	SAMPLES Type	SPT Blows per 6 inches	Moisture Content, %	Dry Density, pcf	Unconfined Compressive Strength, psf	Water Level Depth, feet	Graphic Log	SOIL DESCRIPTION
0									Select low plasticity clay with sand, gravel, and cobbles Stiff to hard consistency
	1	ST		10.9	126	16520			
	2			11.7	120	16930			Class 10, clayey sand with cobbles
	3			10.6	125	9980			
5	4	ST		10.4	114	6160			Select low plasticity clay with sand, gravel, and cobbles, moist Stiff to hard consistency
10	5	ST		10.5	118	8560			
	6	ST		21.8	114	14470			
15	7	ST		13.5	122	13210			
	8			17.1	118	9270			
	9	ST		15.8	114	7260			
									<<B-4>> Boring drilled to 19.2 feet at auger refusal

Civil and Construction Engineering Department - Iowa State University

LOG OF TEST BORING

Boring Number B-5　　Sheet 1 of 1　　**Lab Number** 5

Date Drilled	10/20/97	Project	US Highway 163 Bypass
Surface Elevation			Jasper County - Prairie City
Depth Drilled	23.0 feet	Client	Iowa Department of Transportation
Drilling Method	Hollow-Stem Auger		

Depth to Water _____ ft @ completion(▽), _____ ft @ _____ hrs. (▼), _____ ft @ _____ hrs. (▽)

Depth, feet	SAMPLES Number	SAMPLES Type	SPT Blows per 6 inches	Moisture Content, %	Dry Density, pcf	Unconfined Compressive Strength, psf	Water Level Depth, feet	Graphic Log	SOIL DESCRIPTION
0	1					*5000			Light tan to gray high plasticity clay
	2	ST				*4000			Class 10, clayey sand, low to medium strength
5	3	ST		14.1	118	530			
	4					*2000			
10	5	ST		16.5	108	1000			
	6	ST		16.1	115	3200			
	7			12.0	121	6300			
15	8	ST		13.6	119	3500			Light tan to gray high plasticity clay, very moist, medium strength
	9			27.3	94	2700			
	10			27.5		*3000			
	11	ST		27.5	93	2200			Class 10, tan clayey sand, very wet, medium strength
	12			28.5	94	2100			
20	13	ST		31.4	91	1300			Light tan to gray high plasticity clay, very wet, medium strength
	14					*2000			
	15	ST		29.7	91	2300			Class 10, tan clayey soil, stiff consistency
	16			19.9	106	6500			

<<B-5>> Drilled to 21.8 feet, sampled to 23.0 feet

* Calibrated Penetrometer